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CHANNEL ADJUSTMENTS

VERIFICATION OF SEDIMENT TRANSPORT FUNCTIONS

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ABSTRACT

A comparison of twelve sediment transport functions has been made to study their predictive accuracy. The results are compared with those obtained by the Modified Einstein Procedure (MEP). The study is based on 97 field measurements of sediment and hydraulic variables in Missouri River. In comparing the results, whenever possible, both the bed material concentration and its particle size distribution are considered. Despite considerable scatter based on overall assessment, Toffaleti's method yields best results.

INTRODUCTION

This paper summarizes the findings of a comprehensive study (Mahmood, 1980) on the evaluation of twelve sediment transport functions in predicting the sediment transport rates using the data collected on the Missouri River. Besides the bed material load, the median particle size and the gradation coefficient of the transported bed material were included in the comparison studies, wherever possible. The conclusions reported herein are based on the analysis of 97 field measurements conducted during the period July 1969 - July 1978 at Omaha and Nebraska City gaging stations.

The direct measurement of bed load in field channels is almost impossible without physically altering the flow and sediment regimes around the samplers. Of the indirect procedures, the sequential profiles method offers great advantages over others, due to its innocuous conception, ease of field measurement and overall economy. A set of 39 sequential profiles scanning the Missouri River bed between I-480 and Union Pacific Crossings, therefore, were also analyzed to yield the average bed load.

The computational procedures investigated in this study include the Modified Einstein Procedure (1955), Einstein's Bed Load Function (1950), Mahmood's Transport Function (1971), Toffaleti's Method (1969), Colby's Procedure (1964), The Modified Colby Method (1979), Meyer-Peter and Muller's Formula (1948), Engelund-Hanson's Formula (1967), Acker-White's Function (1973), Yang's Stream Power Relation (1973), Laursen's Function (1958) and Shen-Hwang's Method (1979). After preliminary investigations, Engelund-Hansen's Formula and Yang's Relations were excluded from further study, because they showed relative insensitivity to the input hydraulic and sediment variables.

FIELD DATA AND CLASSIFICATION

All of the data analyzed in this study was collected on the Missouri River. The measurement details and historical notes on the measuring stations can be found elsewhere (Keown et al, 1977). A combination of depth-integrated (DI) and point-integrated (PI) samples are available for estimating the suspended sediment load. Based on the nature of the availability of data, the entire data set is classified into three categories as described below:

Category A: - The measurements available in this category include the volume, sediment weight and particle size distribution of composited DI-samples; bed material particle size distribution; nominal discharge; local flow depth and the temperature of air and water.

Category B: - This category includes both PI- and DI-samples for suspended load. The DI-samples were obtained at all verticals, while the PI-samples were recorded at alternate verticals. Each PI vertical contained five measurement points.

Category C: - The data in this category were collected on the unobstructed channel reach. The suspended load measurements comprise solely the PI-samples. The measurements include the location of verticals, sampling points, point velocity, PI-concentration, its sediment size distribution, particle size distribution of the bed material; nominal discharge and the water temperature.

The range of variables covered in the above data is:

Discharge (m^3/s)	=	480 - 2000
Water Surface Width (m)	=	117 - 218
Hydraulic Depth (m)	=	2.3 - 5.3
Average Velocity (m/s)	=	0.67 - 2.02
Water Temperature (°c)	=	1 - 31
Bed Material Size:		
D_{50} (mm)	=	0.17 - 0.72
gradation σ	=	1.19 - 3.57

In addition to the aforementioned data, sequential bed profiles covering a segment of the Missouri River near Omaha were also observed, and have been analyzed in subsequent studies as a means of evaluating bed material load movement.

TRUE VALUE AND MEASURES OF ERROR

The comparison or verification of functions requires a measure of the standard of true value of the function. The direct measurement of true bed material load in sand bed field channels is presently impossible. The value obtained by the Modified Einstein Procedure (MED) is, therefore, taken as the standard for comparison. The reason for its selection lies in the fact that a large part of the total bed material load is directly measured in this procedure. Only a small layer close to the bed remains unprobed by the sampling device. In the data used herein, the unmeasured depth amounts to Ø.5 ft for DI-sampler and Ø.9 ft for the PI-sampler.

For Category C data, and for some runs of Category B data, the PI-samples are changed into representative DI-samples before estimating the true bed material load by the MEP. For the Missouri River data analyzed in this study, the unmeasured bed material load on the average amounts to 4Ø% of the total bed material load.

Definition of error: - For comparison purposes, the following definitions for error are used

$$\varepsilon_i = f_i - \hat{f}_i \tag{1}$$

where f represents either the predicted value of the total bed material load, or the mean particle size of the bed material load, or its gradation coefficient. The hat (^) indicates the "true value" obtained by the MEP and the subscript, i, refers to the i-th experiment or measurement run. The root-mean-square error is defined as

$$\varepsilon = \left[\frac{1}{n}\sum_{i=1}^{n} \varepsilon_i^2\right]^{1/2} \tag{2}$$

where n represents the total number of field experiments. In addition, the relative error

$$r_i = \frac{f_i - \hat{f}_i}{\hat{f}_i} \tag{3}$$

and its first two moments

$$\mu_r = \frac{1}{n}\sum_{i=1}^{n} r_i \tag{4}$$

$$\sigma_r = \left[\frac{1}{(n-1)} \sum_{i=1}^{n} (r_i - \mu_r)^2\right]^{1/2} \tag{5}$$

are also used.

DISCUSSION OF RESULTS

For comparison purpose, the bed material load, G, and its particle size distribution were calculated, whenever possible. In alluvial channels, the particle size distribution can be well represented by the lognormal probability distribution (Mahmood, 1973). The bed material size distribution, therefore, can be described concisely by its mean diameter, d_{50}, and gradation coefficient.

$$\sigma = \frac{1}{2}\left[\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}}\right] \tag{6}$$

where D_{84}, D_{50}, and D_{16} are 84, 50 and 16 percent finer size, respectively.

Whenever possible, all three of these variables -- G, D_{50} and σ -- were compared with the values obtained by MEP.

Comparison of DI-Measurements With MEP Results:- Figures 1a and 1b are prepared to show the comparison of actually measured bed material concentration by the DI-Sampler with the total bed material concentration computed by the Modified Einstein Procedure. The abscissa in Fig. 1a represents the measured bed material concentration by the U.S. D-49 Sampler. The ordinate represents the total bed material concentration computed by the MEP. It is apparent from this figure that the two concentrations are well correlated in statistical sense. The best-fit line can be represented by

$$\hat{C} = 3.266\ C_m^{0.881} \tag{7}$$

where, C_m = the material concentration obtained by DI-measurements and $\hat{C}$ = the concentration corresponding to the MEP result. Both concentrations in Eq. 7 are expressed in parts per million (ppm). The lower and upper envelopes are shown by dotted lines in Fig. 1a. The curves plotted in Fig. 1b represents the proportion of bed material load which remains unmeasured during DI-sampling. The abscissa represents the total bed material concentration as obtained by the Modified Einstein Procedure. By examining these figures, it is clear that the unmeasured proportion of total bed material concentration decreases as the overall sediment transport increases. For example, on the average, 50% of the bed material concentration remains unmeasured when the total bed material concentration is 100 ppm. This ratio of the unmeasured load to the total load reduces to 38% as the bed material transport increases to 600 ppm (See Fig. 1b). It is emphasized that these figures are valid for DI-measurements using the U.S. D-49 Sampler only, with a 0.5 ft of the flow depth from the bottom remaining unprobed, and for the range of data used in this study.

Einstein Bed Load Function Versus MEP: - A study was made to compare the total bed material loads predicted by the Einstein Bed Load Function and the Modified Einstein Procedure. The results are shown graphically in Fig. 2.

The ordinate in this figure represents the total bed material load, G_e, predicted by the Einstein Bed Load Function and the abscissa represents the total bed material load computed by MEP. It is apparent from this figure that the Einstein Bed Load Function predicts lower values than those predicted by the MEP when the overall sediment transport is not very large. For large sediment transport rates, the Einstein Bed Load Function gives higher values than those obtained by the MEP.

Prediction of Bed Material Load: - A summary of predictive accuracy of different sediment transport functions is given in Table 1. Among all the sediment transport functions which yield both the total bed material load and its particle size distribution, Toffaleti's method gives the minimum rms value of error for Data Categories A and B. For Data Categories B and C, Shen-Hwang's method, which was derived from similar data on Missouri River, gives the minimum error. Among those methods which yield only the med material load and not its particle size distribution, Acker-White's method yields the minimum error. A graphical comparison of Toffaleti's method is shown in Fig. 3, in which the ordinate represents the total bed material load predicted by Toffaleti's method and the abscissa represents the corresponding values obtained by the MEP method.

Prediction of Bed Material Size Distribution: - Only six of the twelve functions investigated in this study can yield particle size distribution of the bed material load. Among these, Toffaleti's method provides the best distribution of particle size. A summary of comparative errors is provided in Table 2. The comparison of mean particle size D_{50} and gradation coefficient, Ø, predicted by Toffaleti's method, with those obtained by the MEP, is shown in Fig. 4 and 5.

Bed Load Prediction: - Of the twelve functions investigated herein, only five yield bed load. Since bed load calculations in these functions are either closely related, or the procedures were established on the basis of almost same data, it is not very meaningful to compare their predictive accuracies among themselves. The actual bed load measurement obtained by sequential profiles can provide a separate and independent check for verification. However, at the time of this study, the sequential bed profiles were not observed frequently enough to cover a wide range of discharge or other variables. During bed profile observation, the discharge in Missouri River in the observation reach remained constant at 50,000 cfs. In Category A, there were eight field experiments in which the discharge in the river, also, remained at roughly 50,000 cfs. These eight data sets were analyzed for comparison purpose. Based on this limited analysis, it was found that the Modified Einstein Procedure produced the maximum (24.4 ppm), and Shen-Hwang's method the minimum (5 ppm) results for bed load, against the observed value of 14 ppm by sequential profile analysis. Due to limited information, it is not possible to arrrive at any definitive conclusion regarding the bed load calculations.

CONCLUSION

The following conclusions are drawn from the analyses made herein:

1. Among the methods that predict the bed material load and its particle size distribution, the two methods that best fit the observed data are Toffaleti's and Shen-Hwang's. Toffaleti's method better fits the MEP-based estimates and Shen-Hwang's, the PI-data-based estimates. However, the particle size distribution for bed material load is better predicted by Toffaleti's method in both cases.

2. Among those functions which yield bed material load only, Acker-White's method gives the best predictive accuracy.

3 Particle size distribution of bed material load is best predicted by Toffaleti's method for all categories of data.

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Table 1

PREDICTIVE ACCURACY OF BED MATERIAL LOAD

METHOD	Bed Material Load and Particle Size									Bed Material Load Only					
	Toffaletti			Laursen			Shen-Hwang			Acker-White			Colby		
	ε	μ_ε	σ_ε	ε	μ_ε	σ_ε	ε	μ_ε	σ_ε	ε	μ_ε	σ_ε	ε	μ_ε	σ_ε
Data	1000 T/D	%	%	1000 T/D	%	%	1000 T/D	%	%	1000 T/D	%	%	1000 T/D	%	%
Category A (MEP)	<u>26.8</u>	-8.3	44.3	53.0	-77.5	10.7	45.5	-59.3	18.1	29.7	15.1	46.5	47.1	59.1	65.1
Category B (MEP)	<u>39.4</u>	35.4	173.8	48.4	-69.9	29.6	42.1	-35.2	57.4	<u>35.1</u>	51.1	157.5	38.1	89.5	145.9
Category B&C (PI)	29.9	58.7	180.2	25.5	-62.3	36.7	<u>20.2</u>	6.7	106.5	<u>31.6</u>	93.8	224.7	44.2	158.0	254.7
Category B&C (ADI)	32.0	83.8	198.0	20.6	-57.3	37.8	<u>17.0</u>	4.9	105.4	<u>34.4</u>	120.2	229.0	47.8	189.9	251.0

Table 2

PREDICTIVE ACCURACY OF PARTICLE SIZE DISTRIBUTION

METHOD	Laursen			Toffaletti			Colby (Modified)			Shen-Hwang		
	ε	μ_ε	σ_ε	ε	μ_ε	σ_ε	ε	μ_ε	σ_ε	ε	μ_ε	σ_ε
Data	mm	%	%	mm	%	%	mm	%	%	mm	%	%
Category A (MEP)												
D_{50}	0.02	-2.1	11.2	<u>0.02</u>	-2.3	10.4	0.04	20.3	12.5	0.04	9.9	17.9
σ	0.14	1.1	9.4	<u>0.12</u>	-3.3	7.0	0.17	-9.1	5.2	0.20	5.8	12.1
Category B (MEP)												
D_{50}	0.02	6.8	12.1	<u>0.02</u>	6.5	11.1	0.05	31.6	11.7	0.04	11.3	20.9
σ	0.15	9.2	5.4	<u>0.09</u>	3.6	5.1	0.13	-7.1	5.7	0.22	13.8	7.5
Category B&C (PI)												
D_{50}	0.03	15.0	13.9	0.03	12.4	12.5	0.08	45.8	20.8	0.04	18.2	22.7
σ	0.36	14.5	14.1	0.03	5.8	13.0	<u>0.29</u>	-1.1	14.7	0.42	19.5	15.8
Category B&C (ADI)												
D_{50}	0.04	11.0	15.6	0.04	8.8	15.8	0.07	40.3	19.0	0.05	14.1	22.8
σ	<u>0.25</u>	6.3	10.6	0.30	-1.6	11.6	0.30	-8.2	10.3	0.31	11.1	14.2

Note: Underscoring indicates minimum value among methods of same type.

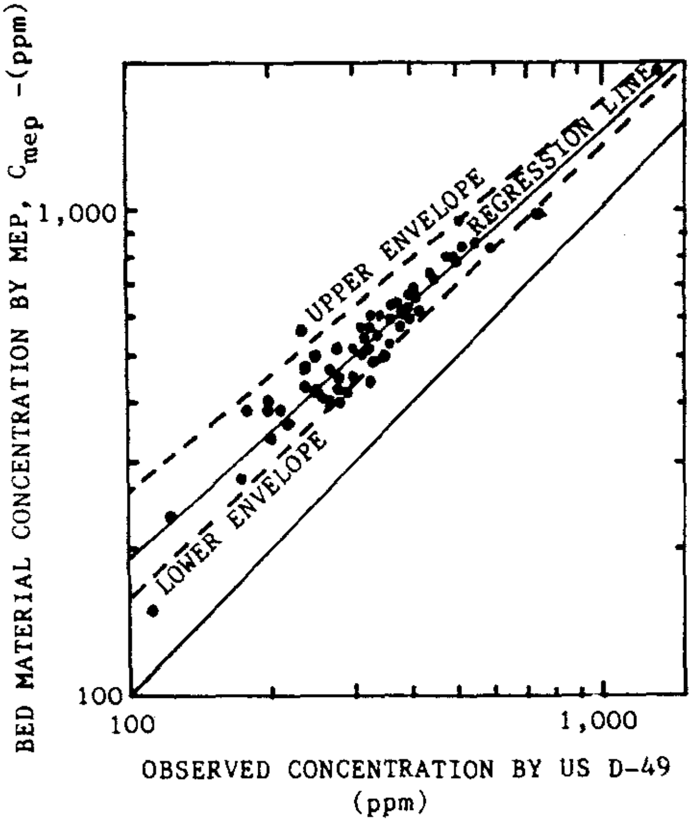

Fig. 1a

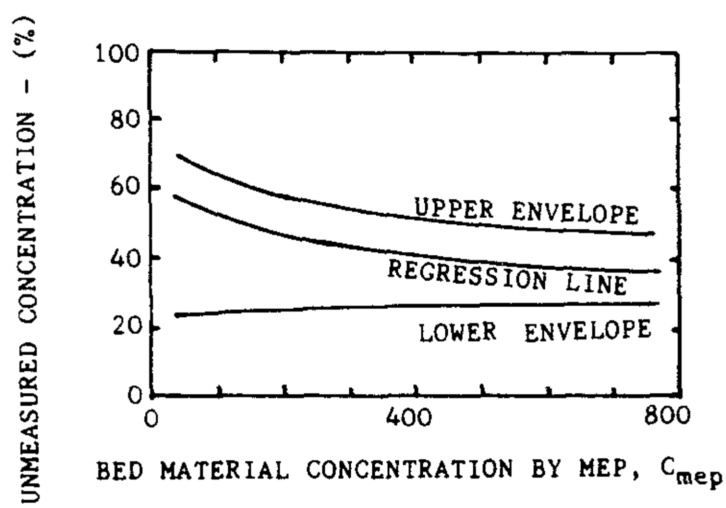

Fig. 1b

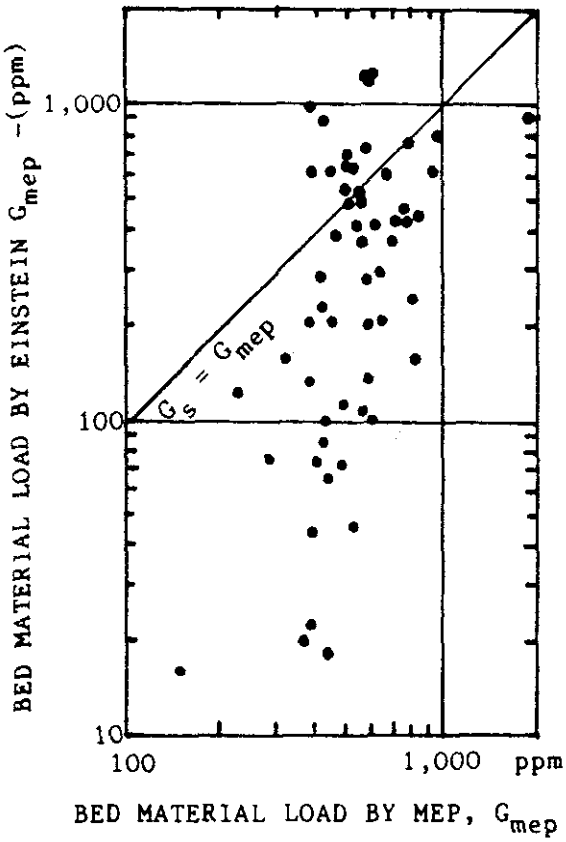

Fig. 2

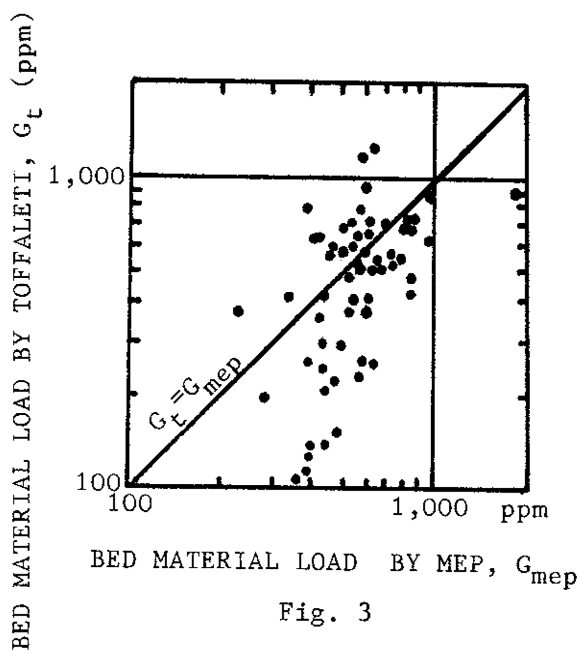

Fig. 3

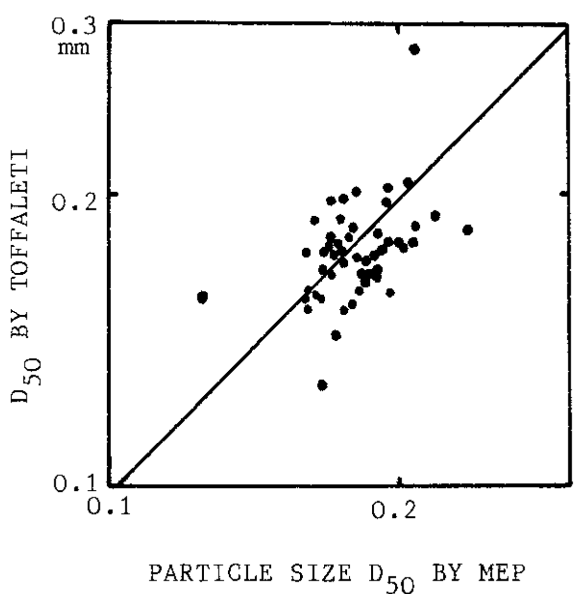

Fig. 4

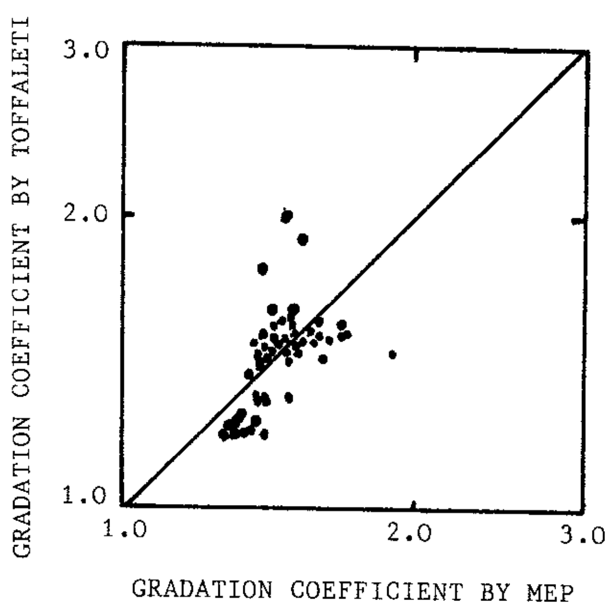

Fig. 5

PHOTOGRAMMETRIC ANALYSIS OF CHANNEL ADJUSTMENT

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ABSTRACT

Comparison of features recorded from successive stereopairs of aerial photographs allows measurement of river-channel migration rates, analysis of changes in channel morphology, and mapping of flood-plain sedimentation patterns. In digital photogrammetry, points and linear features are converted to x, y, and z coordinates; digitized features can be displayed and compared at any scale and in any combination desired. Measurement precision may be as fine as 1-2 m for photographs with scales of approximately 1:20000 if image quality is good and coordinate systems are carefully matched.

An analytical stereoplotter is being used to measure channel changes that occurred between 1971 and 1983 at a site on Smoky Creek, a tributary of the New River in Tennessee. Preliminary results indicate that (1) channel widening was associated with deposition of coarse sediment in the channel; (2) channel changes occurred throughout the 12-year period and at least three events caused significant adjustments; (3) initiation of channel adjustment predated strip mining in the Bowling Branch basin just upstream but may have been influenced by strip mining further upstream in the Smoky Creek basin; (4) aggradation on the flood plain opposite Bowling Branch occurred after the onset of strip mining in the Bowling Branch basin.

INTRODUCTION

River channel geometry and channel pattern often change over time in response to natural or man-induced alterations in hydrologic regime and sediment load. In order to understand the spatial extent and evolutionary sequence of such changes it is necessary to obtain information about the condition of the channel at different points in time. Most published studies of channel adjustment over time rely on repeated field surveys as the primary source of information, but aerial photographs and historical maps have been used as well (Martinson, 1983; Hooke, 1977; Eschner, Hadley, and Crowley, 1983). Aerial photographs at scales commonly available in most of the United States cannot attain the resolution and precision of detailed field surveys, but they can be used to measure changes as small as several meters, and measurements of change are not limited to pre-selected monitoring sites. Aerial photographs at favorable scales are generally available for multiple dates within a time period that extends from the late 1930's to the present, and they record a large amount of surface detail that may be helpful in interpreting processes and sequences of events.

This paper focuses on the use of photogrammetry for measuring channel changes from aerial photographs. The first part of the paper introduces the method and discusses the reliability of photogrammetric measurements of geomorphic processes. The second part of the paper describes preliminary results of photogrammetric analysis of channel change at a site in the Smoky Creek basin in east Tennessee.

AERIAL PHOTOGRAPHS AND PHOTOGRAMMETRY

Vertical aerial photographs made with a calibrated mapping camera record surface detail with a high degree of geometric accuracy. Although aerial photographs commonly are used in analysis of landforms and landform changes over time, the geometry of the photographic image is fundamentally different from the geometry of the topographic maps with which it is often compared, and special procedures must be followed if precise measurements are desired. An aerial photograph is a perspective projection of the ground surface, and image points are subject to horizontal displacements caused by terrain relief and camera tilt. Using a stereoplotter one can combine two overlapping aerial photographs (hereafter referred to as a stereopair) to create an accurate three-dimensional image of the ground surface from which tilt and relief displacement have been removed. Analogue stereoplotters are used in transferring information directly from aerial photographs to base maps by optical projection or by mechanical projection (Wolf, 1983). Martinson (1983) used an analogue stereoplotter to map channel changes occurring along the Powder River in Montana during the period between 1939 and 1978. Estimated accuracy of the erosion measurements, averaged over the length of the channel reach measured, was 6.6 m of lateral displacement.

DIGITAL PHOTOGRAMMETRY

In digital photogrammetry the problem of capturing data from aerial photographs is solved through mathematical rather than through optical or mechanical means. Accuracy and flexibility of operation are greatly enhanced with the assistance of computer technology. Points and linear features traced with a stereoplotter are converted to x, y, and z ground coordinates and can be displayed in any combination and at any scale desired. Measurements of temporal change are made using computer algorithms that compare spatial locations of features digitized from photographs taken on different dates.

The success of a photogrammetric mapping project depends on the availability of a network of ground control points that can be used to establish the spatial coordinates of features identified on the aerial photographs. A control point may be any feature that is clearly visible and that can be located with pinpoint precision on the aerial photograph, provided that its horizontal coordinates and its elevation are known. Typical features used for ground control include center points of road intersections, fence-line intersections, corners of buildings, rock outcrops, or other stable natural features. Ground coordinates of each control point and digitized locations of the corresponding point visible in each photographic image of the stereopair are used as input to an analytical aerotriangulation program. The program solves a set of equations to provide the correct parameters for a coordinate transformation between the image and the ground. Redundant data are used to perform a least squares adjustment, and residual errors are helpful in identifying problems and assessing the accuracy of the results.

In order to measure river-channel changes and other geomorphic changes using digital photogrammetry, it is necessary to register aerial photographs of different dates to the same ground coordinate system. Ideally all ground control points should be surveyed in the field and identified by targets that are visible in at least one set of aerial photographs. In addition it is necessary to have a series of supplementary points, known as passpoints, that form well-defined images on all photographs used in the study and that can be used to transfer control from one stereomodel to another.

ACCURACY

Planimetric accuracy of point location on the photograph is approximately 20 to 40 micrometers. For photographs with a scale of 1:24000, the accuracy of point location is equivalent to 0.5 to 1.0 m on the ground. Meier and others (1985), using digital photogrammetry to monitor flow velocities and strain rates on the Columbia Glacier, found a photogrammetric accuracy of about 3 m for displacement vectors of well-defined points on photographs with a nominal scale of 1:46000.

If field survey data are not available it is possible to use other information sources to determine ground control coordinates. Horizontal coordinates of well-defined points on U.S. Geological Survey topographic maps can be determined with the help of a digitizer. A small number of points on each quadrangle map are annotated with spot elevations; elevations for other points can be determined by interpolation between topographic contours. The National Map Accuracy Standards can be used to estimate the errors associated with control points obtained from topographic maps (Slama and others, 1980, p.372). Although the values of the control-point coordinates are subject to errors inherent in the map, these errors do not significantly affect the scale of the digitized features and their estimated magnitude is not indicative of the precision of horizontal measurements of change over time. When the goal is to obtain a precise match of coordinate systems between successive stereopairs for the specific purpose of measuring change, it is possible to use map points to find a coordinate transformation for a single stereopair and then to use output coordinates of points in the first stereopair as input coordinates for the same points in a second stereopair. Using this method, Miller (1983) found that residuals of control points commonly were about 1 m in the x- and y-directions and 0.5 m in the z-direction. Experiments conducted to verify horizontal measurement precision yielded a sample standard deviation of less than 1 m in measuring displacements of sharply defined linear features from photographs with scales of 1:20000 to 1:30000. The size of the error was larger for natural features that were less sharply defined and for photographic images of poor quality. Despite the relatively good results found for horizontal measurements of change, vertical displacements typically are quite sensitive to small errors in elevation of ground control points. Elevation comparisons therefore should not be considered reliable unless verified by field survey or by independent measurements of stable points common to both stereopairs.

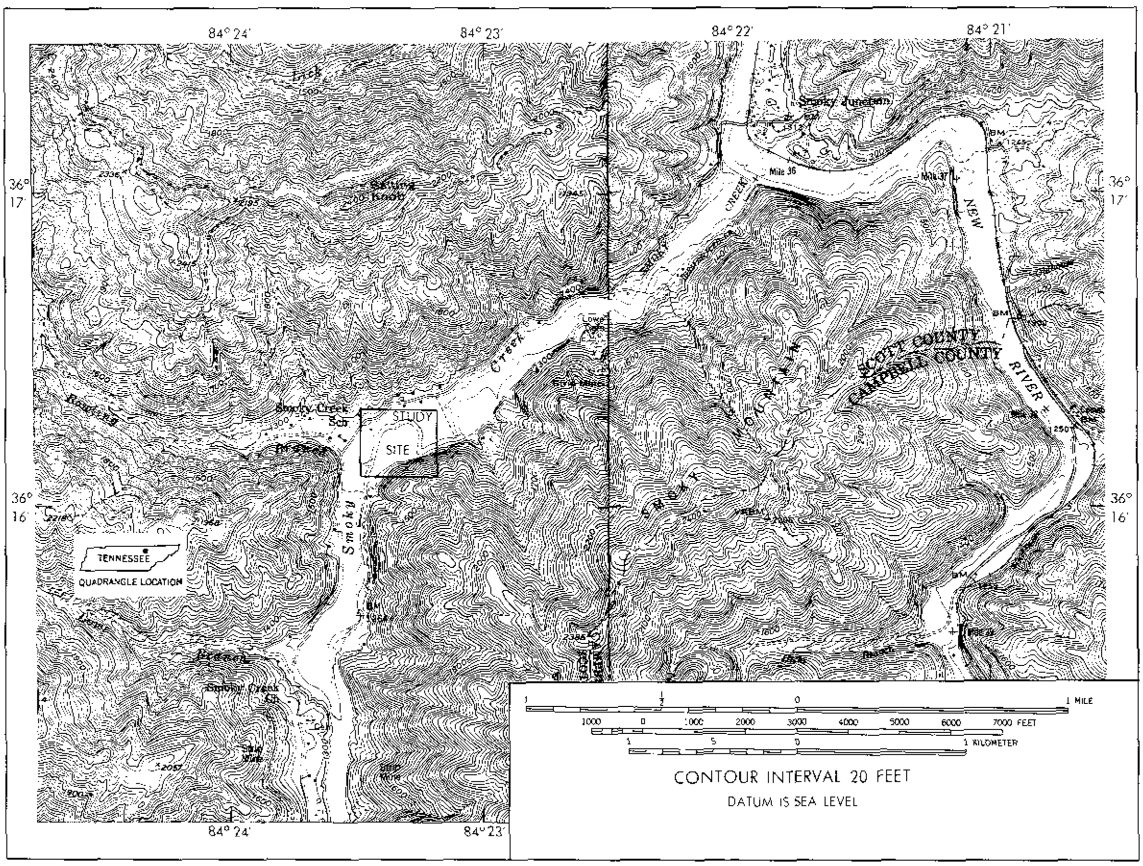
PHOTOGRAMMETRIC MEASUREMENT OF CHANNEL CHANGES ON SMOKY CREEK

Smoky Creek, a tributary of the New River in northeastern Tennessee, was chosen as a site for photogrammetric investigation of channel change because rapid channel adjustment to land disturbance by strip mining in the Smoky Creek basin has occurred in recent years. Studies are currently in progress to quantify rates of channel migration, to map changes in channel width and patterns of erosion and sedimentation, and to delineate features on the flood plain that have been formed by overbank flow. Aerial photographs are available for several dates within the period of rapid channel change; detailed photogrammetric analysis will help to resolve questions about spatial patterns of channel response to upstream disturbance and about the timing of channel adjustment. Although the current study relies primarily upon analysis of aerial photographs, it is expected that this analysis will prove most useful in conjunction with the results of ongoing field investigations (B.A. Bryan, U.S. Geological Survey, personal communication, 1985.)

The Smoky Creek basin has a drainage area of 85 km^2. A U.S. Geological Survey gage (drainage area 44.6 km^2) at Hembree, about 4 river miles upstream from the study site, was operated between 1976 and 1983. Records of several years' duration are available for several other gages on tribtaries of Smoky Creek. All of these tributary stations have been discontinued.

The basin, located within the Cumberland Plateau physiographic province, is underlain by Pennsylvanian shales with intercalated siltstones, sandstones, and coal beds (Avery and Luther, 1970). The rocks in the vicinity of the study site are flat-lying and the coal seams are subject to extensive contour mining on mountain slopes (Gaydos and others, 1982). Slopes typically are steep, averaging from 20 to 60 percent, and alluvial valleys are narrow and deep (figure 1).

Figure 1. Reference map of the study area



Surface mining has a dramatic impact on sediment load and channel morphology in the Smoky Creek basin. Osterkamp and others (1984) used botanical and sedimentological evidence to demonstrate that channel widening, gravel bar formation, incision of channel deposits, and rapid lateral migration have occurred in recent decades on Smoky Creek and on several of its tributaries in drainage basins where strip mining has occurred. Channel disturbance is attributed to the introduction of large amounts of coarse sediment from mined basins and to increased peak discharge from tributary basins, and aggradation of the Smoky Creek flood plain is observed below mined tributary basins.

The study site is a meander and adjacent flood plain on Smoky Creek just below Bowling Branch, a tributary basin with a drainage area of 7.7 km^2 (figure 1). The meander marks an abrupt change in orientation of the valley of Smoky Creek from north-south to northeast-southwest, and also marks a change in channel sinuosity. Along the reach extending from Bowling Branch upstream to Hembree, the sinuosity of Smoky Creek is 1.09. Along the reach between Bowling Branch and the mouth of Smoky Creek, the sinuosity of the channel is 1.35.

Mapping is being done on an analytical stereoplotter at the U.S. Army Engineering Topographic Laboratories in Fort Belvoir, Virginia. Control points for use in aerotriangulation were digitized from U.S. Geological Survey topographic quadrangle maps. Experiments to determine precision of the results have not yet been completed, but the results described by Miller (1983) suggest that the standard deviation of measurement error should fall in the range of 1 to 5 m of horizontal displacement. Interpretations made below are based on preliminary results of photogrammetric mapping. The sequence of changes mapped is illustrated by comparing enlarged portions of aerial photographs made on four different dates (figures 2a, 2b, 2c, and 2d; see table 1 for a listing of photographs used in the study). Index numbers in the text refer to annotations on the appropriate photograph.

Table 1. Reference data for aerial photographs used in this study

Date	Source	Frame index numbers	Nominal scale
11-13-71	TVA[1]	81246, 81247	1:20000
10-18-76	TVA	128343, 128344	1:20000
10-17-77	TVA	139686, 139687	1:20000
3-07-83	TVA	2090-181, 2090-182	1:24000

1 Tennessee Valley Authority, Mapping Services Branch, Chattanooga, Tenn

CHANNEL CHANGES ON SMOKY CREEK AT BOWLING BRANCH

Bowling Branch enters Smoky Creek from the west (figure 2a, location 1); immediately below the tributary junction there are small sand bars located on both sides of the channel. Opposite the junction are two prominent flood channels and several subsidiary drainage lines that extend all the way across the flood plain (location 2) in a direction nearly perpendicular to the channel of Smoky Creek. The meander pattern below (north of) Bowling Branch consists of two discrete bends without a reversal of curvature between them. The outer bank just below the center of the first bend is sharply defined and appears to be eroding (location 3). At the entrance to the second bend, a prominent bar extends across the channel from the outer bank (location 4). Erosion caused by diversion of flow toward the inner bank has created a distinct notch just upstream of the axis of the bend (5). Below the axis of the bend, the bank is vegetated on both sides and appears stable. Photogrammetric evidence of major channel instability is confined to the area described above; no features of comparable importance are visible between here and the mouth of Smoky Creek.

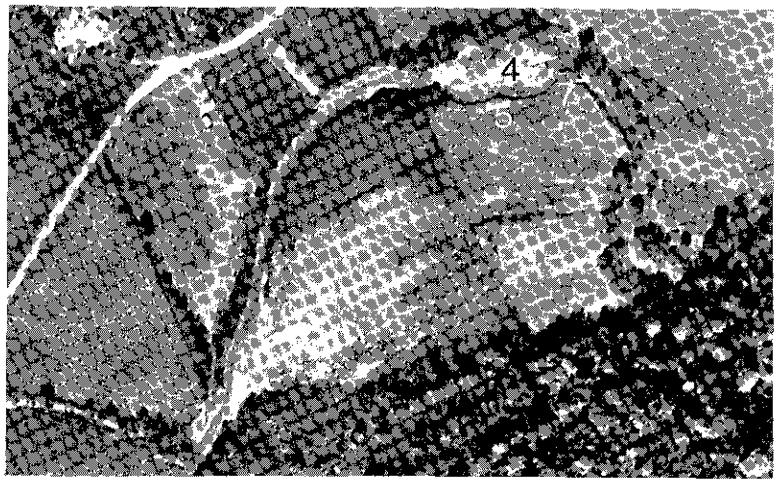
Comparison with the photograph taken 5 years later in 1976 (figure 2b) shows that the bars at the mouth of Bowling Branch (1) are much larger than before, and that the channel splits around one of these bars. Splay deposits extend up onto the floodplain opposite the tributary junction (2). Digitized features indicate minor erosion at location 3 and point bar deposition on the inner bank opposite this location. The bar complex at 4 has expanded rapidly toward the inner bank, and erosion associated with diversion of flow around the bar (5) has widened the belt between the inner bank and the tree line marking the edge of the bar complex on the outer bank (6). This belt is now about 3 times as wide as the stable channel upstream and downstream on Smoky Creek. Some accumulations of sediment are visible in the channel downstream of this point.

The next aerial photograph, taken one year later, shows additional changes (figure 2c). The bars below the mouth of Bowling Branch (1) have been reworked. The longitudinal bar in the center of the channel is gone and the flow follows a single thread close to the left bank; accretion has occurred along the right bank. Some of the surface relief associated with channel features on the flood plain opposite the mouth of Bowling Branch has been obscured by aggradation (2). Bank erosion has progressed further at location 5 and widening of the channel has occurred as far downstream as location 7; the bar complex (4) shows some evidence of dissection and reworking, but the appearance of the bar surface may also be affected by slight differences of water stage in the channel between figures 2b and 2c.

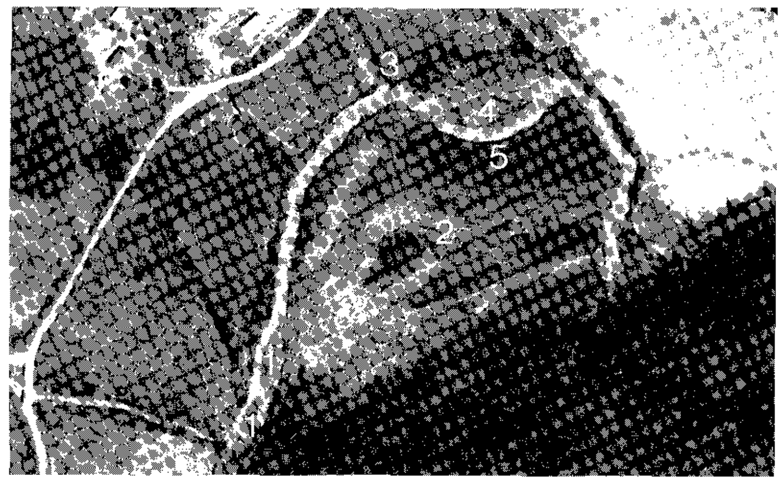
Comparison of figure 2c with figure 2d, taken in 1983, reveals continuing change. Deposition of gravel below the mouth of Bowling Branch has formed new bars in the channel, and accretion is continuing along the right bank (1). Extensive aggradation has occurred on the floodplain (2); the flood deposits shown here may have been left behind after a cloudburst in late summer of 1982 that affected Bowling Branch but was not detected upstream on Smoky Creek (B.A. Bryan, U.S. Geological Survey, personal communication, 1985.) Photogrammetric mapping of these deposits may be used as a guide in field investigations to determine distribution of sediment thickness and volume and mass of aggraded material. Further bank erosion has occurred at locations 3 and 5, and the formerly irregular pattern of bar growth and channel widening by bank erosion appears now to have developed the smooth curvature of a convex point bar (4) and concave bank, with a chute cutting across the point bar. Completion of photogrammetric mapping will allow estimation of net change in the amount of sediment stored in the floodplain and the bar complex.

a. November 13, 1971

b. October 18, 1976



c. October 17, 1977



d. March 7, 1983

Figure 2. Aerial photographs of the study site (see figure 1 for location). All photographs are oriented with north at the top. Numbers indicate features referred to in the text.

DISCUSSION

Visible adjustments of the Smoky Creek channel at the study site strongly suggest that the impacts of strip mining described by Osterkamp and others (1984) at other sites upstream in the basin have also been felt here. Deposition of coarse material in the channel diverted the main filament of flow toward the inner bank and caused persistent rapid erosion during the period 1971-1983; evidently this process began before 1971 (figure 2a). Flood plain aggradation by increased sediment loads has obscured much of the relief that was visible in 1971, and the change after 1976 is particularly noticeable.

Flood flows clearly play an important role in mobilization of coarse sediment, formation of gravel bars, and bank erosion, but the sequence of changes cannot be attributed to a single extreme hydrologic event. The peak discharge of record on Smoky Creek (249 m^3 per second at Hembree) occurred on April 4, 1977 and was 2.5 times larger than the second highest peak flow for the period of record (beginning in December 1976). The aerial photographs shown in figures 2b and 2c were chosen for study because they bracketed this event in time, and it was expected that most of the major channel changes for the period of the study would be associated with the large sediment load carried by this flood. Although there were substantial changes, these changes fit into a continuum of channel adjustments as illustrated in figure 2. Cumulative changes between 1971 and 1976 and between 1977 and 1983 were larger than changes that occurred during the year in which the flood of record was measured. Even if all change measured during each period is attributable to a single flood event, there were at least three significant channel-forming events during a 12-year period. Such events may have been less frequent prior to widespread strip mining in the Smoky Creek basin, but there are no gage records available to resolve this question.

Osterkamp and others (1984), working on Smoky Creek below the mouths of mined tributary basins, attributed gravel-bar storage of tractive load to the local tributary source and used the mass of sediment stored in bars and floodplain deposits as a rough indicator of the yield of coarse sediment from mined tributary basins. The situation at the Bowling Branch site is made somewhat more complex by the chronology of strip mine development in the Bowling Branch drainage. Aerial photographs show that no mining or other large-scale slope disturbance had occurred by November of 1971. By 1976 a network of roads and trails had been built on the slopes in the northwest part of the basin, but no mines had yet been opened and no evidence of slope failure or accelerated erosion is visible in the aerial photographs. The 1977 photographs show a small area of open strip mine near the ridge crest in the northwest corner of the basin, and by 1983 the area of strip mining was much larger and clearly was a major source of sediment. Accelerated floodplain aggradation after 1976 probably was due to increased sediment yield from Bowling Branch. Osterkamp (1985, personal communication) reports that the surficial deposits visible on the flood plain in figure 2d were examined in the summer of 1984 and were found to contain large concentrations of coal, whereas underlying sediments were relatively poor in coal. This observation provides corroborative evidence that local flood-plain aggradation was associated with the onset of strip mining in Bowling Branch.

Although much of the sediment deposited in the channel and on the floodplain after 1976 was derived from Bowling Branch, photogrammetric analysis shows that bar growth and channel widening downstream of Bowling Branch began earlier and, if related to strip mining, must have been caused by deposition of coarse sediment derived from strip mining further upstream in the Smoky Creek basin. The origin of the bars visible immediately below the mouth of Bowling Branch in 1976 (location 1, figure 2b) may be attributable to slope changes in Smoky Creek caused by sediment emerging from Bowling Branch or to backwater effects of flood flows from Bowling Branch that caused deposition of sediment arriving from further upstream. The location of the bar complex at location 4 in figure 2 may be a function of channel planform rather than proximity to the sediment source. The bend just downstream of this bar has a much smaller radius of curvature than the first bend below Bowling Branch. As the ratio of radius of curvature to channel width decreases, resistance to flow in the bend increases rapidly (Bagnold, 1960). Resistance to flow in a tight bend can cause backwater effects immediately upstream, reducing the rate of energy expenditure on the bed of the stream (Ippen and Drinker, 1962). The initiation of growth of the bar complex may have occurred when flood flows on Smoky Creek first began to carry large amounts of coarse debris derived from strip mining elsewhere in the basin. Some of this material probably was reworked from channel deposits and gravel bars that formed immediately below the mouths of mined tributary basins. After a flood flow carrying a large tractive load traversed a long, relatively straight reach, the first meander bend tight enough to generate backwater effects would be a natural locus for channel aggradation and widening.

According to Church and Jones (1982, p.313), "in some gravel-bed rivers, parti-cularly extensive bar assemblages develop at certain locations where far greater volumes of coarse sediment are stored than elsewhere along the channel... Sedimentation zones are areas of channel instability: the intervening reaches, termed by us 'transport reaches', may remain stable while the wave [of bedload material] passes through, suggesting that most of the material moves fairly quickly from one sedimentation zone to the next." The unstable portions of the study site described above and the relatively stable local reaches upstream and downstream on Smoky Creek appear to fit this description. Sedimentation zones clearly occur at the mouths of mined tributary basins (Osterkamp and others, 1984), and, if the interpretation presented above is correct, they also occur at channel transitions where a sudden change in flow direction affects the capacity of the river to transport coarse material. Field studies are currently in progress to identify channel transitions controlling formation of gravel bars in the New River (B.A. Bryan, 1985, personal communication).

SUMMARY

River-channel adjustments to the effects of human disturbance may be quite rapid, and patterns and rates of adjustment are ideal subjects for photogrammetric analysis. Although some of the changes illustrated in this study are quite dramatic, precise measurement techniques are needed in order to quantify the range of process rates that may be of interest. The use of digital photogrammetry minimizes measurement error and makes it possible to measure change in geomorphic features to within a few meters using aerial photography at scales that are commonly available. Adequate ground control and careful matching of coordinate systems from one stereo-pair to another are critical elements that affect the reliability of such measurements.

The example of Smoky Creek illustrates the types of changes that may be documented with this method. Comparison of digitized features may be used directly or in combination with field investigations to quantify rates of erosion, accretion, and lateral migration, and to estimate changes in sediment storage over time. Because the aerial photographic record extends back to the 1930's, it may be possible to compare changes occurring before and after disturbance by strip mining began to affect the channel system. Application of digital photogrammetry will thus enhance our understanding of the cumulative impact of strip mining on fluvial systems.

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GEOMORPHIC - HYDRAULIC SIMULATION OF CHANNEL EVOLUTION

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ABSTRACT

Major incision following channelization, with concomitant channel widening and soil loss, has required rehabilitation of several north central Mississippi channels.

Previous geomorphic and hydraulic research by the authors identified a process-based conceptual model of channel evolution which used location-for-time substitution. This was necessary because data was only available for surveys widely separated in time.

A numerical simulation procedure based on geomorphic and hydraulic models was used to quantify changes in morphology and channel response. This allowed a year-by-year prediction of soil loss and channel change, which provides information for efficient planning for channel rehabilitation programs. The numerical simulation utilized hydraulic sediment routing to determine channel aggradation and degradation. Bank stability estimates and channel morphologic relationships were used to simulate channel widening. The results of the simulation closely corresponded to channel dimensions 15 years after channelization.

INTRODUCTION

Intensive agricultural development of north central Mississippi since 1830 has resulted in significant changes in the runoff and sediment yield of watersheds within the region (Watson, 1982). In the 1960's, conservation measures that reduced watershed sediment supply to the channels and channelization for flood protection resulted in an imbalance between sediment supply and sediment transport capacity (Schumm et al, 1981). The net result has been major incision of many channels with related channel widening and soil loss (Schumm et al, 1984).

Channel incision predisposes the channel to widen by increasing gravitational forces on the banks (Little et al, 1982). Schumm et al (1981) suggested that the failed banks and degrading channel beds are the principal sources of bedload-sized sediment in these degraded channels. Median bed-material size is on the order of .012 inch to .02 inch (0.3 mm to 0.5 mm) for streams in the North-Central Plateau subdivision of the Gulf Coastal Plain Physiographic province (Fenneman, 1932). These streams do not have a gravel source.

During previous geomorphic and hydraulic studies of Oaklimiter Creek, Schumm et al (1981) developed a design procedure which significantly reduced the cost and number of structures in the rehabilitation of that channel. The geomorphic approach resulted in allowable slopes that are less conservative than those required by the modified regime and tractive stress methods (SCS, 1977).

Complete surveys for Oaklimiter Creek are only available for the years 1965 and 1981. Therefore, little is known about conditions in the interim and the effect of hydraulic, geomorphic, and hydrologic parameters on the channel evolution. Sediment yield has been estimated from comparison of channel cross-section surveys at two points in time. Further, with no procedure to simulate the process of channel incision and widening, useful evaluation of design alternatives is limited.

OBJECTIVE

The objective of this investigation was to develop a numerical simulation procedure to quantify the movement of sediment in aggrading or degrading reaches of the channel and also to quantify the widening of the channels as the evolution process occurred. This allowed estimates of soil loss and concomitant channel widening and degradation to be predicted for the periods between surveys and for the future.

LITERATURE REVIEW

Schumm et al (1981, 1984) and Harvey et al (1983) report on the 15-year response of Oaklimiter Creek to channelization that occurred in 1965. Oaklimiter Creek is located in Benton County, northern Mississippi and it has a drainage area of 42 square miles (Fig. 1). Oaklimiter Creek was channelized in order to reduce the frequency of out-of-bank flows which were reported to occur about 10 times per year. Prior to 1965 there had been very little modification of the natural channel, which had sinuosities ranging from 1.3 to 2.5.

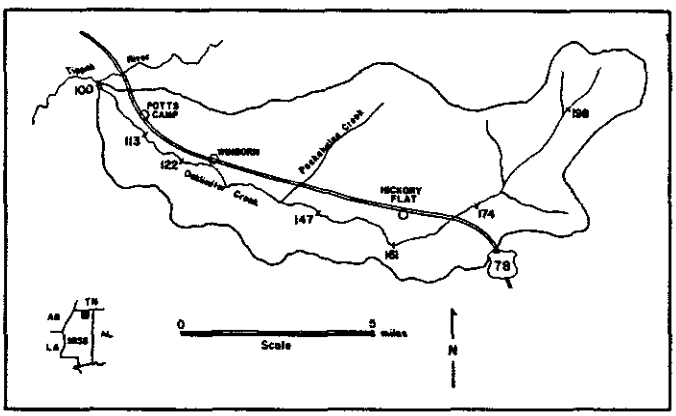

Fig. 1 Location map of Oaklimiter Creek Watershed. Cross section locations are indicated.

As Oaklimiter Creek incised, slope decreased. Slope reduction would have been greater but resistant Tertiary-age outcrops and several man-made structures reduced incision and overfalls occurred as resistant material was encountered. This resulted in a stepped channel profile with the mean gradient reduced by 52% after 15 years. The reduction in slope was accomplished by bed degradation which incised channel depths by an average factor of 2.83. The increased channel depths caused bank failures, which resulted in channel widening, by an average factor of 2.66. The combination of increased depth and width resulted in significant increases in cross-section area.

An evolutionary sequence of channel cross sections was identified in the field. The use of a location-for-time substitution technique enabled development of a channel evolution model (Fig. 2) composed of five channel types (Types I - V) (Schumm et al, 1981). The channel types are associated with conditions ranging from total disequilibrium to a new state of quasi-equilibrium. Figure 2 schematically depicts the evolution of the channel reaches through time, and Table 1 provides a summary of the changing magnitude of the variables and the dominant processes, through time. In Oaklimiter Creek a state approaching quasi-equilibrium was recognized (i.e., Type IV reach) when (1) the width-depth ratio (F) was about 6, (2) channel slope had reduced to approximately 0.001, (3) sediment accumulation in the bed of the channel had reached about 3 feet, and (4) a meandering thalweg had developed under low-flow conditions.

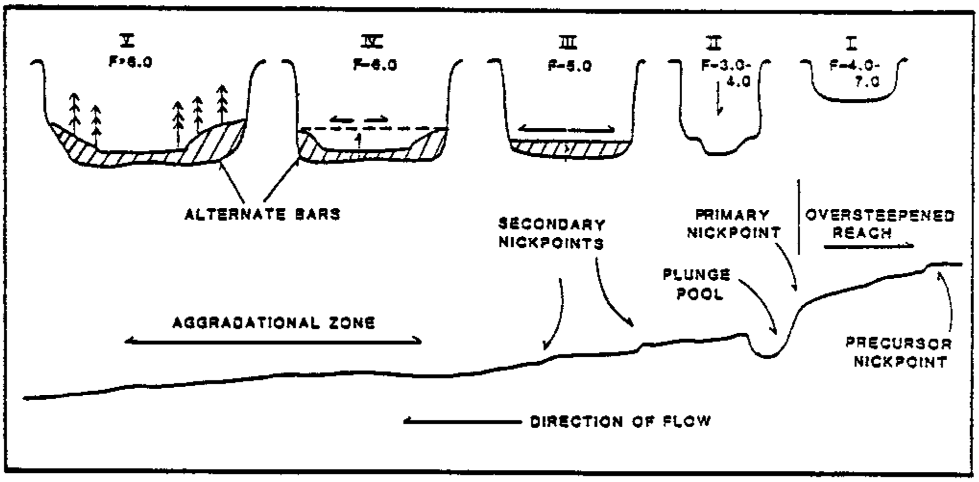

Fig. 2 Schematic longitudinal profile of an active channel showing identifiable features. Schematic cross section profiles corresponding to reaches on the longitudinal profile show the evolution of the reaches from Type I to Type V. Typical width/depth ratio (F) values are shown.

Table 1 Summary of morphometric data used to determine channel evolution model, Oaklimiter Creek.

STAGE (1)	LOCATION (2)	TOP WIDTH in feet (3)	DEPTH in feet (4)	WIDTH/DEPTH RATIO (5)	THALWEG SLOPE (6)	DEPTH OF SEDIMENT in feet (7)	DOMINANT PROCESS (8)
I	Upstream of nickpoint	82	17.3	4.7	0.0020	0	Transport Sediment
II	Immediately downstream of nickpoint	82	21.6	3.8	0.0018	variable	Degradation
III	Downstream of II	100	20.1	4.9	0.0018	1.5	Rapid Widening
IV	Downstream of of III	115	19.2	6.0	0.0016	2.5	Aggradation and Development of Meandering Thalweg
V	Downstream of IV	119	15.3	7.8	0.0010	6.3	Aggradation and Stabilization of Alternate Bars

NOTE: 1 ft = 0.305 m

Oaklimiter Creek therefore appears to have responded to channelization in the same fashion as other channelized streams (Daniels, 1960; Barnard, 1977; Schumm et al, 1984). Schumm et al (1981) and Harvey et al (1983) carried their efforts beyond those of the other investigators when they attempted to use the channel response to date to predict the future development of the non-equilibrium reaches. Their predictive relationships were based on two assumptions: (1) that base level for Oaklimiter Creek would remain at its current elevation and (2) that land use in the watershed would not significantly change.

As hydrologic and hydraulic data became available for Oaklimiter Creek a relationship between top width and stream power was developed for the 5-year discharge:

$$T.W. = f(Q_5 \cdot S_5) \tag{1}$$

This relationship is similar to that of Henderson (1961) and Wolman and Brush (1961) which demonstrated that the wetted perimeter of the channel increases with stream power. Stream power was defined as the discharge-slope product.

Watson and Harvey (1984) utilized data from Schumm (1961) to extend the relationship functionally to include the effect of bed and bank materials in sixteen stable reaches of four streams in semi-arid western USA (Eq. 2). Hydrologic and hydraulic information was not available for these streams, therefore a surrogate parameter for stream power was used. Area-gradient Index (AGI) is the product of drainage area and channel slope, which are highly correlated with discharge and hydraulic gradient (Park, 1977; Simons and Senturk, 1977). Utilizing the AGI, the D_{50} of the bed material, and the weighted mean percentage of the bed and bank material (M), a least squares, multi-linear regression was developed, using metric units, for channel top width in quasi-equilibrium reaches:

$$T.W. = 53.2\ AGI^{.26} D_{50}^{.24}\ M^{-.16} \tag{2}$$

The correlation coefficient (r) for the regression is 0.91. Each of the independent variables was also highly correlated with top width, having respective correlation coefficients of 0.82, 0.83, and -0.85.

These relationships estimate the quasi-equilibrium top width for specific channels. However, the goal of this numerical simulation procedure is to provide a means of estimating channel morphology as it evolves, thus the previous methods were not used and an effort was made to model the physical processes involved in the conceptual model of channel evolution. Bank failure is the physical process controlling channel widening.

Thorne et al (1981) and Little et al (1982) investigated bank failures along an incised channel in north-central Mississippi. They collected field data to describe critical bank heights and slope angles. They concluded that the dominant mode of bank failure was a slab failure in which tension cracking was an important component. Failure was observed to occur along a slip surface between the toe of the slope and the bottom of the tension crack. Development of tension cracks appeared to be related to repeated wetting and drying of the sediments, which have a high montmorillonite content (Harvey et al, 1983). Thorne et al (1981) used a log-spiral failure surface to model bank failure. By establishing values of bank height and slope angle they were able to estimate critical stability for a given bank section for conditions without tension cracks and for an aged bank with the tension cracks. The tension crack depth was assumed to be a maximum of half the total bank height (Terzaghi, 1943).

SIMULATION PROCEDURE

To simulate the channel evolution process adequately, the numerical procedure must incorporate hydraulic routing of sediment, the strength of the bank materials that resist gravity failure, and a procedure to estimate the channel widening after thresholds of bank strength are exceeded.

An existing sediment routing computer program was selected (Simons et al, 1981). This computer program uses HEC-2 for the water surface profile calculations. A sediment transport rate is then computed at each cross section and the transport rate is budgeted to estimate aggradation or degradation in successive channel reaches.

Channel widening cannot occur until the aged critical bank height is exceeded by degradation of the bed. Original construction depths were less than aged critical bank heights, and thus, degradation continued without channel widening until this bank height threshold was exceeded.

Critical bank height (h_c) and aged critical bank height (h_c') was estimated using the Culmann method, assuming vertical banks. Mean values of soil strengths were obtained for each stratigraphic unit using data from nearby Hotophia Creek (Thorne et al, 1981). The critical bank height and the aged critical bank height were then computed for each cross section location utilizing previous stratigraphic mapping along Oaklimiter Creek (Schumm et al, 1981).

The channel widening model for the simulation is based upon the conceptual model of channel evolution (Fig. 2). That conceptual model (Schumm et al, 1981) utilized a location-for-time substitution technique. The use of the technique means that channel widening that will occur through time at one location can be predicted by obtaining data from other locations. This approach was employed for the simulation process, and a least squares multi-linear regression was used to establish an empirical relationship for change in top width as a function of channel depth and of characteristics of the five channel phases.

The data show that channel widening rates varied considerably during the evolution process. Data from six repeat surveys on nearby Topashaw Creek provided by the Soil Conservation Service, U.S.D.A., showed that widening began at a very low rate and then increased rapidly to 14.36 feet/year in six years. Eleven years later the rate was only 1.21 feet/year.

Oaklimiter Creek channel morphology data obtained in 1980, included cross-section surveys, depth of aggraded sediment, local thalweg slope, and descriptors of channel planform and stratigraphy. A channel evolution type (i.e., I through V) was assigned to each of the sections, and these data were used in the statistical analysis to establish an empirical relationship for change in channel top width.

The width/depth ratio (F) was then included with maximum depth and depth to predict cumulative top width change:

$$\Delta T.W. = 0.0318\ d_{max}^{.385}\ d^{1.423} F^{1.342} \qquad R^2 = .96 \qquad (3)$$

The agreement between observed and predicted cumulative top width change was found to be very good.

The numerical simulation used equation (3) to predict cumulative change in top width. Channel top width change at any time (N) is predicted on the basis of channel morphology at the preceeding time period (N-1), as follows:

$$\Delta T.W._N = f(d_{max(N-1)}\ d_{(N-1)}\ F_{(N-1)}) \qquad (4)$$

The widening increment is the difference between the cumulative change in top width ($\Delta T.W._N$) and the previous cumulative change:

$$T.W.(\text{increment}) = (\Delta T.W._N + T.W._{1965}) - T.W._{N-1} \qquad (5)$$

and the total width at any time is:

$$\text{Total Top Width} = \Delta T.W._N + T.W._{1965} \qquad (6)$$

Figure 3 is a schematic representation of the Oaklimiter Creek simulation procedure. HEC-2 input data describes channel size and slope, initial roughness, and flow rates. The basic HEC-2 water surface profile computation is made thereby providing hydraulic conditions for sediment transport calculation and analysis for aggradation/degradation.

Sediment particle size distribution of the original bed and bank materials is initial input. The sediment is routed by size fraction which results in a coarsening of the bed as finer material is transported from the system. Adjustment of the cross section geometry for aggradation, degradation and/or widening then concludes the time step computations and the HEC-2 cross-section data are updated to reflect these changes. Results for each time step include channel size, sediment transport rates, hydraulic conditions, and volume of eroded or deposited material.

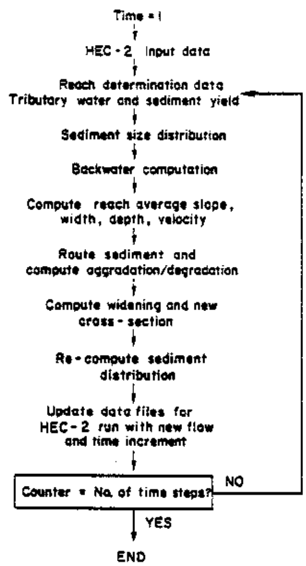

Fig. 3 Schematic of the Execution Sequence of the Numerical Simulation

RESULTS

The two primary results of the simulation process were: (1) demonstration of the ability to predict changes and damages which may be caused by channel degradation and widening, and (2) increased understanding of the channel evolution process.

The mean elevation of the 1981 surveyed profile and the results of simulation up to 1981 differed by less than one foot, and maximum variation was 4 feet. The actual mean top width of Oaklimiter Creek as surveyed in 1981 was 99 feet and the mean top width of the simulated channel up to 1981 was 91 feet. Prediction of the top width was accomplished within 10% of actual top width. Figure 4 shows the cumulative sediment yield from the numerical simulation. A previous estimate (Schumm et al, 1981) of soil loss from just the channel based on comparison of surveys in 1965 and 1981, was 4,900,000 tons. The numerical simulation predicted about 6,500,000 tons from the total watershed. With this numerical simulation, the soil and land loss can be computed, and the extent of damage to bridges and adjacent structures could be assessed.

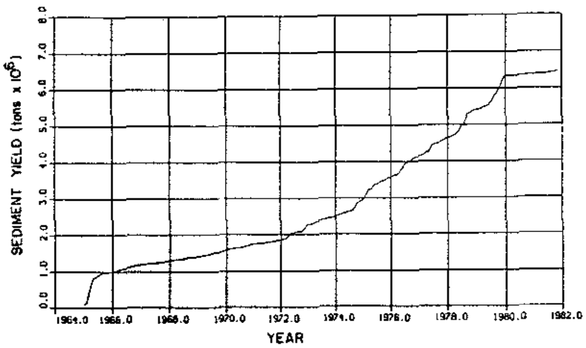

Fig. 4 Cumulative sediment yield from Oaklimiter Creek.

Figure 5 compares the top width and depth changes through time for three cross sections: 122, 147, and 161 (see Figure 1). Following the initial incision after 1965 and large storms in 1973, a series of six sheet-pile and rip-rap sill grade-control structures were constructed on Oaklimiter Creek between sections 122 and 147. As shown in Figure 5, these sections were in an aggradation phase which probably contributed to the success of the structure. Section 161 is well upstream of the structures. This section experienced significant incision from 1976 to 1978, which resulted in severe channel widening.

SUMMARY

A computer program based on geomorphic-hydraulic concepts was developed to numerically simulate the evolutionary adjustment of incised channels (Schumm et al, 1981, 1984). A combination of geomorphology, hydraulics, and geotechnical engineering provides a means of developing a useful planning tool for channel rehabilitation and flood control projects, as well as providing a framework to evaluate present limitations of our knowledge of channel evolution processes. Further research is particularly needed to understand the changes in bank stability during channel evolution.

Acknowledgments

The study of Oaklimiter Creek and development of the simulation procedure was funded by the U.S.D.A., Soil Conservation Service under Contract Nos. SCS-23-MS-80 and SCS-54-MS-83. Mr. Pete Forsythe of the Jackson, Mississippi office and other members of the SCS and ARS have been most helpful in this work.

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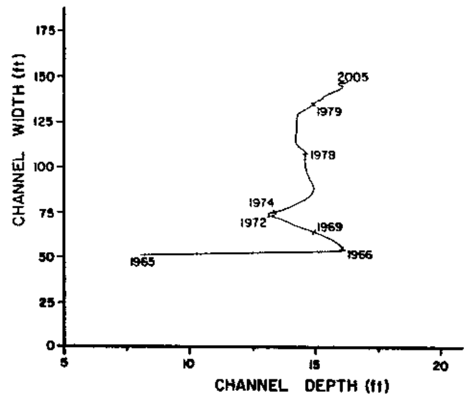

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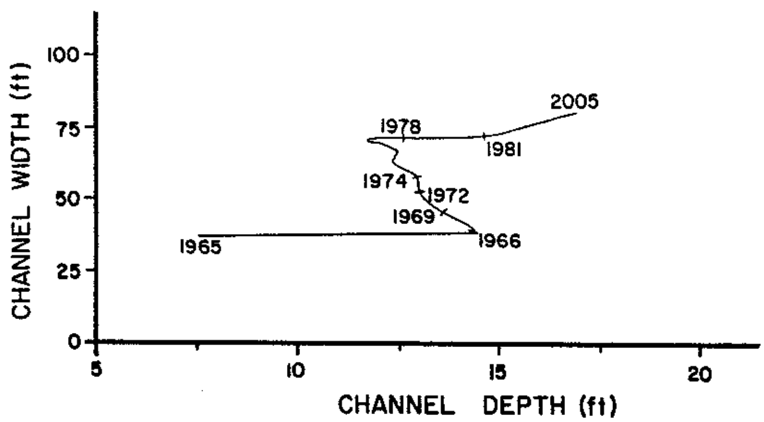

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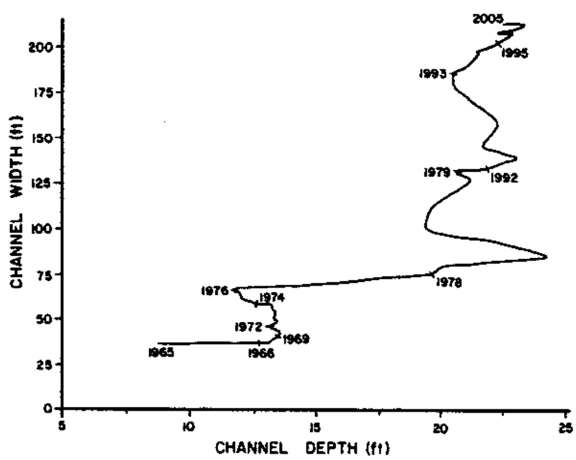

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Fig. 5 Top width and depth change for three Oaklimiter Creek cross sections.

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CHANNEL WIDTH ADJUSTMENT IN STRAIGHT ALLUVIAL STREAMS

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ABSTRACT

Alluvial streams construct their own geometries in response to changes imposed by nature or by man. Channel width adjustment is examined by considering separately vertical and lateral scour and fill. The mechanisms associated with these changes are used to formulate a mathematical model of width adjustment during channel aggradation and degradation. The model, although dependent on approximate assumptions and *a priori* knowledge of certain bank stability properties, reproduces expected behavior with reasonable resolution.

INTRODUCTION

In recent years several water and sediment routing models have been developed for simulating stream channel changes with time as a result of altered water and sediment discharge. These models simulate channel metamorphosis exclusively as a result of longitudinal scour and fill while ignoring channel width changes. Alluvial streams always experience width adjustments either by fluvial erosion or accretion of the banks, or by mass failure of their banks under gravity and subsequent removal of slump debris. Therefore, it is essential that sediment routing models correctly describe the accumulation and depletion of bank and bed material within a channel cross section.

The purpose of this paper is twofold: (1) to give a brief introductory discussion of channel adjustment processes in physical terms, and (2) to provide a quantitative framework for analysis of width adjustments occurring during channel aggradation and degradation. This model is formulated with the aid of several approximate but reasonable assumptions, and its application requires *a priori* knowledge of certain stability properties of bank materials.

RELATION OF CHANNEL GEOMETRY TO HYDROLOGIC REGIME AND BANK STABILITY

The shape of a stream channel is highly organized and is similar for streams of the same size in a comparable climate. Through time, the channel winds through a flat valley (floodplain) bordered by abandoned valley levels (terraces) and hillslopes. The floodplain is constructed by the stream during lateral migration and by deposition of sediment. During this process of lateral movement, the channel maintains a width and depth approximately that of current observations (Leopold et al., 1964).

Because of a change in climate or a change in watershed condition due to agricultural practices, urbanization, or other influences, a new relation develops between discharge and sediment production. The stream may change its level either upward (aggradation) or downward (degradation), building a new level of floodplain appropriate to the new bed elevation.

Channels increase in size downstream as tributaries enter. Therefore, drainage area at any point is closely correlated with mean channel geometry and discharge characteristics. Stream flow largely determines the size of stream channels and amplitude and wavelength of meanders. Frequently bankfull discharge is correlated with the channel size and pattern (Wolman and Miller, 1960). Leopold and Maddock (1953) demonstrated that for most rivers, the mean water surface width and depth of a channel reach increase with mean annual discharge. Of these variables, mean channel width is the most reliable and indicative for describing stream characteristics and morphology. Sediment load is the other independent variable that significantly controls river geometry. Lacey (1930) concluded from an analysis of regime canal data that the channel size is directly dependent on water discharge but channel shape reflects sediment load type. Coarse sediment produces channels with a high width/depth ratio and fine sediment produces narrow and deep cross sections.

During a study of deposition of three ephemeral-stream channels in the western United States it was concluded that the nature of sediment that moved through and deposited in the channel strongly influenced the processes of deposition and the nature of channel erosion (Schumm, 1961; Patton and Schumm, 1981; Schumm et al., 1984). The three channels drain source areas that have high sediment yields ranging from predominantly sand (Arroyo Calabasas, New Mexico) to a mixture of sand, silt and clay (Sand Creek, Nebraska) to largely silt and clay (Sage Creek, South Dakota). The manner by which the channels aggrade and the morphology of the channels is controlled by the sediment size. The wide shallow channel of Arroyo Calabasas is filled by vertical accretion and sand-size sediment. The narrow and deep channel of Sage Creek is created by the lateral accretion of cohesive fine-grained sediment. The Sand Creek channel is filled by a combination of these processes. During the later stages of filling, all the deposited material was silt and clay, with the coarser sands and gravels being deposited farther upstream. These and other similar observations led Schumm and his associates to the conclusions summarized in Table 1.

Leopold (1973) describes similar changes in the channel of Watts Branch near Rockville, Maryland, over a period of twenty years due to urbanization of the watershed. The cross-sectional channel area decreased twenty percent, with approximately seventy percent of this change attributed to the change in channel width. Although aggradation of the streambed did occur, the average elevation of the bed was insignificant compared to the decrease in average channel width (Fig. 1).

Erosion by channel widening is a common occurrence along many miles of streams throughout the United States (Bowie, 1982). This erosion process usually results from scour of the bed and bank toe which increases the height and slope angle of the bank, decreasing its stability with respect to mass failure under gravity. Overheightening and oversteepening on the banks continues until the forces tending to cause failure exceed those tending to oppose failure. The mechanism of failure depends on the height, steepness and stratigraphy of the bank and the physical properties of the bank materials. Different slope stability analyses have been applied to streambanks to determine their stability and to examine the most critical mechanism of failure. An extensive review of these methods has been presented by Osman (1985).

MODELLING CHANGES IN STRAIGHT ERODIBLE CHANNELS

A mathematical model has been developed that computes not only streambed aggradation and degradation, but also accounts for channel width changes resulting from bank erosion and lateral sediment accretion. This model has three major components: water and sediment routing, and changes in bed profile and channel width. The mathematical scheme employs a space-time domain in which the space domain is represented by discrete cross sections along a channel reach, and the time domain is represented by discrete time steps, Δt. Bed material movement and width adjustment processes are regarded as slow in comparison with changes in water discharge and stage. The model takes advantage of this fact by uncoupling the governing equations, treating unsteady flows as piecewise-steady, and by restricting the unsteady part of the computations to sediment routing and bank adjustment processes.

One-dimensional Water and Sediment Routing: The basic equations for water routing include the following momentum and continuity equations:

$$\partial Q/\partial x - q_w = 0 \tag{1}$$

$$(\partial/\partial x)(V^2/2g + y) + S_f = 0 \tag{2}$$

where Q is the water discharge; x is the streamwise distance; q_w is the rate of lateral water inflow per unit length; V is the average water velocity; g is the acceleration of gravity; y is the stage; and S_f is the friction slope. These equations are solved for V and y using the standard step method and prescribed initial and boundary conditions.

Sediment routing is essentially based on the continuity equation:

$$\partial Q_s/\partial x + (1-\lambda)(\partial A_B/\partial t) - q_s = 0 \tag{3}$$

in which Q_s is the volumetric sediment rate; λ is the porosity of the bed material; A_B is the cross-sectional area of bed material storage; t is time; and q_s is the rate of lateral sediment inflow per unit length. The change in cross-sectional area ΔA_B for each cross section at the end of each time step is determined by solving Eq. 3 using a finite-difference scheme with appropriate initial and boundary conditions. Sediment transport rates are computed for different size fractions, and transport capacity is estimated using a formula appropriate for the load characteristics (Alonso et al., 1981b). Deposition ($\Delta A_B > 0$) is controlled by transport capacity, but in the case of channel erosion ($\Delta A_B < 0$), sediment rate is controlled by availability of bed and bank material. Entrainment and composition of bed material are simulated as described by Alonso et al. (1981a).

Distribution of Cross-section Scour: The simulation of channel widening due to bank failure is dependent on lateral bank-toe scour and bed degradation. Scour of the bed and bank increases the height of the bank and the bed width, thus decreasing the stability of the bank. The rate of lateral bank scour depends largely on flow characteristics and the critical shear stress, τ_{Lc}, at which the bank-toe material begins to move. Once this threshold is exceeded by the flow tractive force, τ_0, the bank toe is assumed to retreat at a constant rate R resulting in an increase of bed width $\Delta_B = R\,\Delta_t$ (Fig. 2a). Osman (1985) presents empirical relationships for τ_{Lc} and R that take into consideration bank soil properties. The resulting volume of bank material eroded per unit length of channel, ΔA_L (Fig. 2a), is deducted from ΔA_B yielding the effective depth of bed degradation:

$$\Delta z = (\Delta A_B - 0.5\ (\Delta B)^2 \tan(i))/(B+\Delta B) \tag{4}$$

in which i is the bank slope angle. This depth is used to compute a new bank height, H', and a new effective bank angle $i' = 0.5(i+\theta)$ which are in turn used to check for bank stability. The angle θ is defined in Fig. 2a.

Depending on the size, geometry, and composition of the bank and the strength properties of the bank materials, the banks may experience different types of slope or shear failure. In many instances tensile stresses generated near the top of the bank lead to the development of tension cracks which have an important effect on the overall mode of failure. Contingent upon the mechanism of failure, the analysis of bank stability may be undertaken in a number of ways (Little et al., 1982; Osman, 1985).

For the sake of computational efficiency, the approach adopted in this study was to determine stability of the bank by comparing the new bank height and slope with threshold values obtained from soil strength data. Based on the experimental results presented by Little et al. (1982) bank stability is characterized by a threshold condition relating critical bank height, H_c, to critical slope angle, i_c, as follows:

$$\log H_c = a - b\ i_c; \quad a,b > 0\ . \tag{5}$$

Whenever these critical values are exceeded, failure is assumed to occur along a failure plane ST (Fig. 2a) passing through the bank toe and forming an angle $\beta = 0.5(i'+\phi)$ with the horizontal. The fallen material (polygonal section PRST, Fig. 2a) comes to rest at the foot of the bank as a mass of lose soil standing at the angle of repose, ϕ (Fig. 2b). This fallen material is subsequently removed by flow erosion. During this phase of basal removal, sediment transport capacity is satisfied by the fallen material and no bed degradation is assumed to take place.

This model does not attempt to account for the effects of vertical tension cracks and multiple stratigraphics units. Nevertheless, these effects conceivably could be incorporated in an implicit, though rudimentary, fashion through suitable weighted values of the experimental coefficients in Eq. 5.

Distribution of Cross-sectional Fill: The simulation of sediment deposition in aggrading reaches is controlled by the size of sediment in transport. Coarse material moving as bedload is deposited on the bed starting from the lowest point, and it builds up the channel bed in horizontal layers. Suspended material is deposited on and near the banks resulting in lateral accretion of finer sediments.

The concentration of suspended sediment is typically greatest in the middle part of a stream channel and is least along the banks (Hubbell and Matejka, 1959). The lateral concentration gradient results in a net diffusion of suspended sediment towards the banks where the level of turbulence is lowest, causing the flow near the banks to become overloaded. From this observation Einstein (1972) concluded that suspended sediment is continuously being deposited on the banks of active channels. Based on this hypothesis Parker (1978) postulated a diffusive mechanism for lateral export and arrived at an expression for lateral flux of suspended sediment, F_L, which for the case of net deposition yields:

$$dF_L/dy = -D \tag{6}$$

where D is the rate of deposition per unit area and unit time, and y is the lateral distance. F_L is approximated by the expression:

$$F_L = -\varepsilon_y(d\zeta/dy) \tag{7}$$

in which ζ is the vertically integrated concentration of suspended sediment, and εy is the lateral turbulent diffusivity. The distribution of ζ with respect to y is determined by resorting to formal similarities between the lateral dispersion of suspended sediments and soluble tracers (Alonso, 1981), which enables use of the transport model developed by Yotsukura and Sayre (1976) in the form:

$$\partial\zeta'/\partial x = (\partial/\partial q_c)[h^2\ V\ \varepsilon_y\ (\partial\zeta'/\partial q_c)] \tag{8}$$

in which ζ' and V stand for depth-averaged values of suspended sediment concentration and flow velocity, respectively, and q_c is the cumulative streamflow discharge.

Equations 6, 7, and 8 are solved using implicit finite-difference schemes and boundary conditions requiring vanishing lateral flux and deposition at the water edges. Additional constraints imposed on the solutions are that the average values of ζ' and the cumulative value of D equal the suspended load concentration and the overall rate of deposition, respectively.

SAMPLE MODEL APPLICATION

If the relative complexity of a channel system is measured by the drainage density of the system, then simulation of complex channel networks is largely a problem of computational effort. A hypothetical elementary channel will suffice here to demonstrate the essential capabilities of the present model.

This procedure is applied to study (a) the effect of bed degradation on the stability of stream banks, and (b) the effect of channel aggradation on lateral and vertical accretion. The hypothetical channel is 150-meters long with an initial bed slope of 0.001. The initial cross-sectional shape was trapezoidal with a 15-meter wide bottom and a 70-degree bank slope angle. Bed material is coarse sand having D_{50} = 0.80 mm. The total simulation time was divided into three periods: (1) a first period in which clear water entered the channel at a constant rate of 4.2 m^3/sec sufficient to trigger bed degradation followed by bank failure and channel widening; (2) a second period during which water inflow was increased to 75 m^3/sec and sediment having D_{50} = 0.40 mm was supplied to the channel at a steady rate of 8000 ppm; and (3) a third period during which water inflow was reduced to 10 m^3/sec and coarser sediment with D_{50} = 0.80 mm entered the channel at a constant rate of 800 ppm. The channel roughness was adjusted to attain nearly bankful stage at maximum flow discharge. The combined flow duration for these three periods was 5.7 days.

Channel changes, including those in cross-sectional shape, streambed profile, and channel width are summarized in Figs. 3 through 6. During the run, bank failure occurs at approximately 7 hours. Bed degradatin is active initially, particularly in the upper reaches, but decreases rapidly with time as the channel widens (Fig. 3). The rate at which the bed is lowered is reduced by the sediment supplied by the banks, which contributes toward decreasing the transport capacity excess. At 39 hours into the run most of the failed material has been eroded away, leaving a wider channel with less steep banks in the upper reaches (Fig. 4). Additional simulations not reported for lack of space show that streambed degradation is significantly lower when compared with the degradation of the same channel assuming nonerodible banks.

As the flow increases and fine suspended sediment enters the channel at excess capacity aggradation occurs relatively fast, and sediment is deposited on and near the banks as expected (Fig. 5). During the third flow period, a considerable portion of the coarse sediment entered the channel as bedload is deposited on the bed. Lateral and vertical accretion proceeds downstream building up the banks and the bed, making the banks less steep and thus more stable (Fig. 6). This increased bank stability will conceivably diminish the tendency toward mass failure during subsequent flow events.

This sequence of simulated water and sediment inflows, though arbitrary, represents a probable flow event for the hypothetical channel. The subsequent bank failure, fluvial erosion for the failed material, and channel aggradation demonstrate the model's capability to reproduce these physical processes with a reasonable degree of resolution. The results also illustrate the importance of accounting for stability of streambanks when modeling changes in alluvial streams.

SUMMARY

The general objective here has been a summary discussion of physical processes controlling channel width adjustment, and the presentation of a model as a framework within which those processes may be evaluated.

An alluvial stream builds its own shape; therefore it will respond to any change imposed by nature or by man through self adjustments. Changes in straight channels may include streambed aggradation and degradation and width variations. These changes are closely interrelated as they may occur concurrently, and are heavily dependent on characteristic flow discharges, type of sediment loading, and bank material properties and geometry. Therefore, mathematical models for erodible channels must include these variables.

A model that simulates these changes has been formulated and applied to a hypothetical erodible channel to demonstrate the capabilities of the model. The model shows that the rate at which a channel bed degrades decreases significantly where there is bank failure. Also, it shows that bank accretion during aggradation can lead to increased bank stability.

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Table 1. Types of Channel Adjustment in Straight Alluvial Reaches (after Schumm et al., 1984)

Transport mode and type of bank material	Aggrading channel	Eroding channel
Find bed material and suspended load predominant.	Major deposition of fine material on banks and narrowing of channel.	Streambed erosion predominant after minor initial bank erosion.
Mixed sediment load and bed material.	Major deposition of fines on bank followed by bed deposition of coarser material.	Initial streambed erosion followed by channel widening.
Coarse bed material and bedload predominant.	Streambed deposition predominant.	Channel widening predominant.

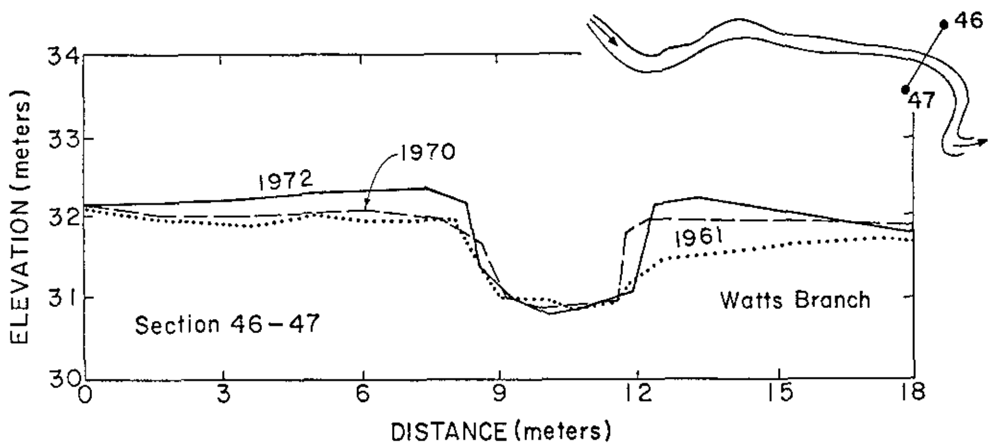

Fig. 1. Selected cross sections of Watts Branch near Rockville, Maryland, over a period of years. Changes are attributed to urbanization (after Leopold, 1973).

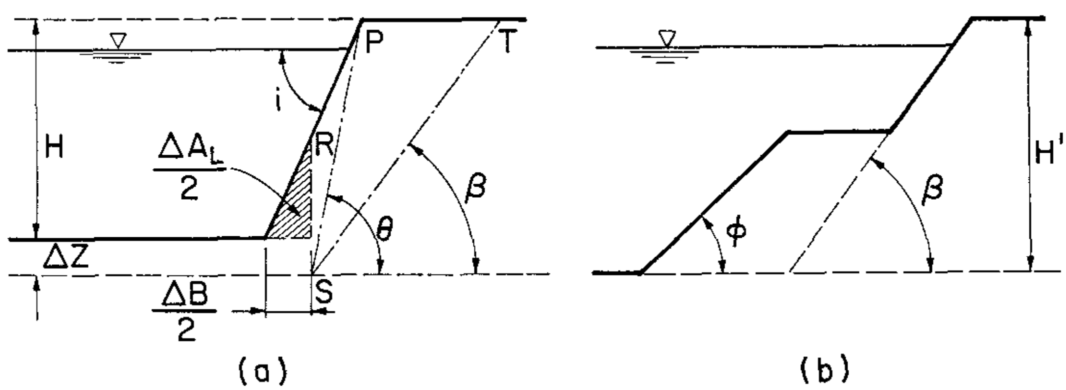

Fig. 2. Definition sketch of stream bank (a) before failure, and (b) after failure.

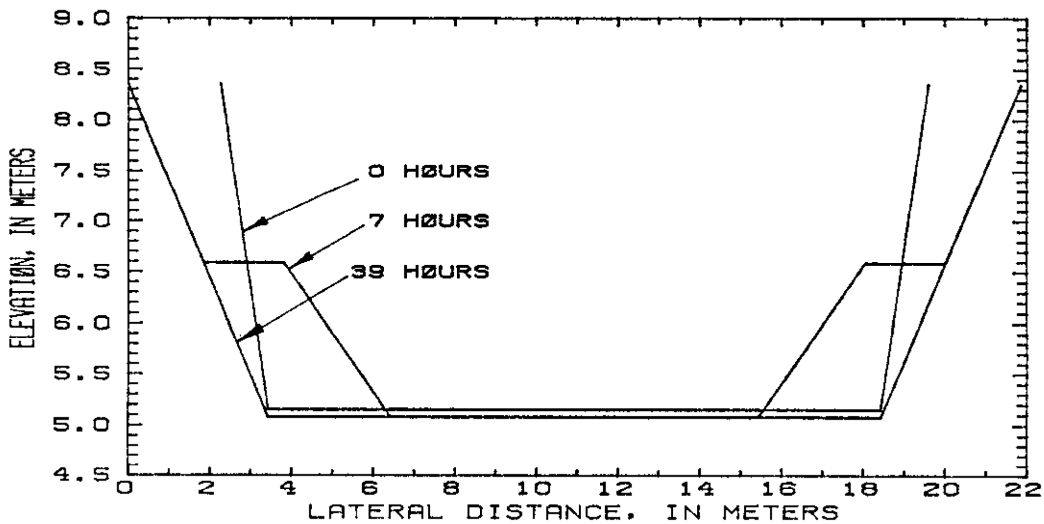

Fig. 3. Cross-sectional changes between beginning of run and end of channel widening period at x = 2 meters.

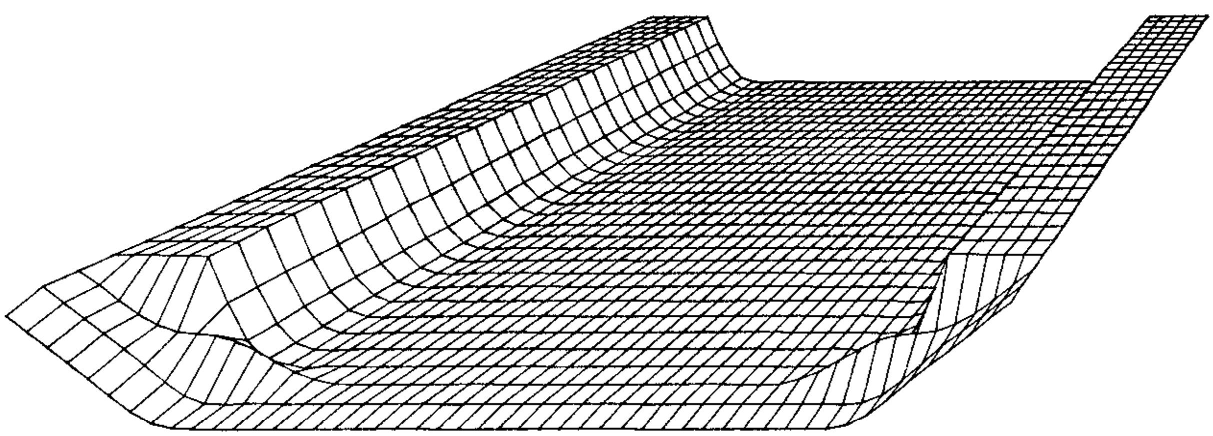

Fig. 4. Channel shape at 93 hours into the simulation run.

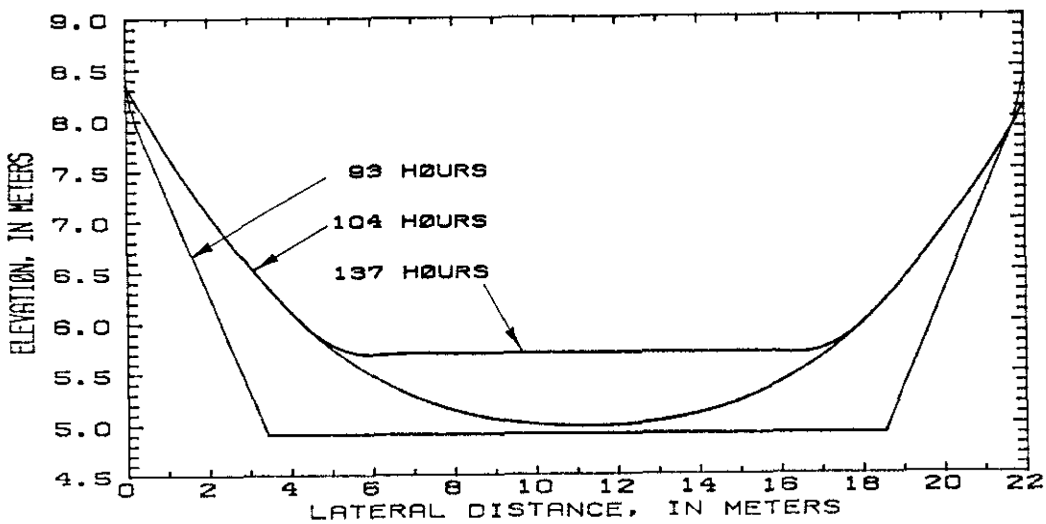

Fig. 5. Changes in cross-sectional shape between 93 and 137 hours into the run at x = 2 meters.

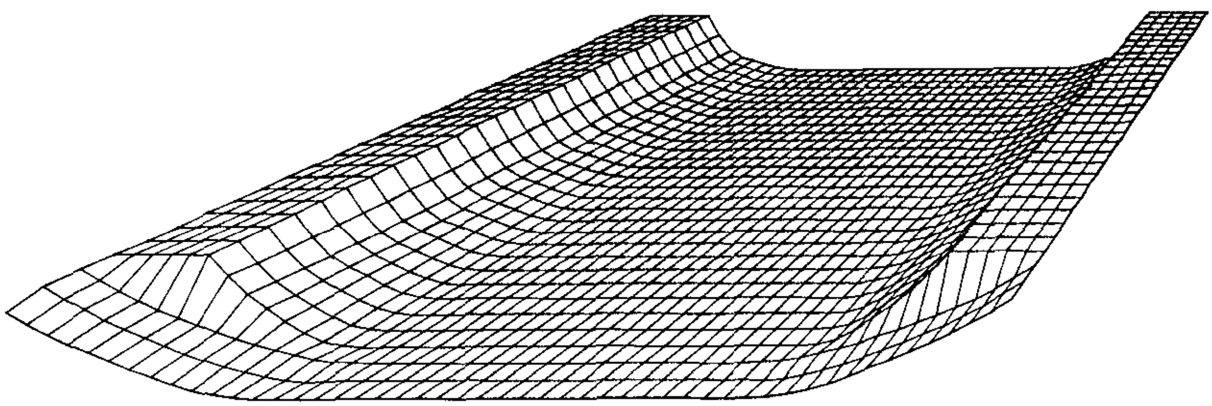

Fig. 6. Channel shape at 137 hours into the simulation run.

PREDICTING CHANNEL ADJUSTMENT TO CHANNELIZATION

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ABSTRACT

A geomorphic study of Muddy Fork was conducted to determine whether the bed and banks would require armoring if channelization were implemented to reduce the incidence of out-of-bank flooding. The study showed that the entire channel was in a state of disequilibrium but that it could be divided into an upper eroding reach and a lower relatively stable reach on the basis of the cohesiveness of the perimeter sediments. This was quantified by two separate multivariate least squares regression relationships between channel top width, stream power, d_{50} and percent silt-clay (M) in the perimeter sediments. Comparison of the ratio of existing to allowable tractive stress in Elk Creek, which had been channelized in 1962 without adverse results, with the two reaches of Muddy Fork confirmed that the lower reach of Muddy Fork did not require armoring, but that the channelized portion within the upper reach would require rip-rap on the banks. Armoring of the bed is unnecessary because it is controlled by out-crops of shale.

INTRODUCTION

In 1964 a project was devised to alleviate flood damage in Muddy Fork watershed, which is located in a four-county area in southern Indiana (Fig. 1). Out-of-bank flows were reported to occur on the average of three times per year. The project was designed to reduce overall flood damage by 90% and the acreage flooded during the 5-year frequency storm by 89% (SCS, 1964). The project proposed land treatment measures to reduce runoff and sediment yield, construction of 3 single purpose floodwater-retarding structures, construction of 4 multi-purpose floodwater-retarding structures, and channelization of the lower 12.5 miles of

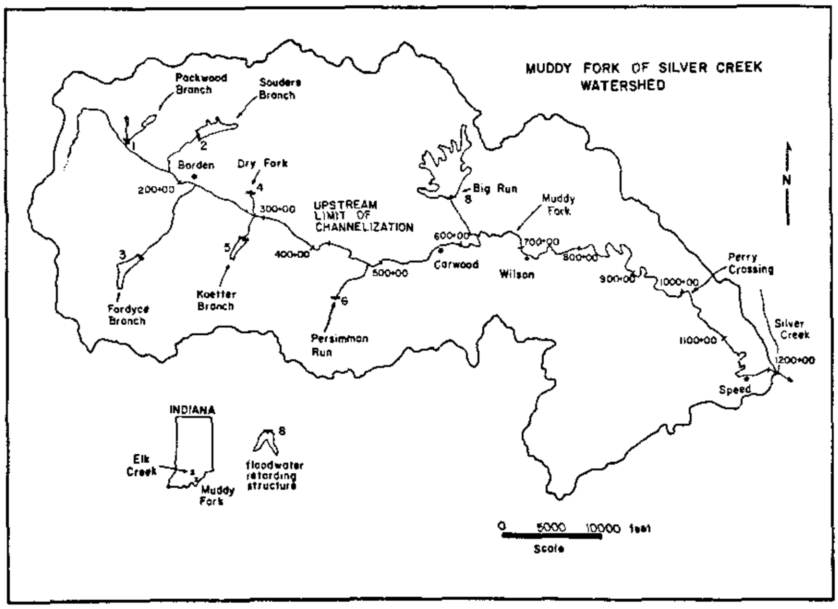

Fig. 1. Location map for Muddy Fork, Silver Creek.

Muddy Fork. To date, five of the structures have been built (Nos. 1, 2, 3, 5, 8 - Fig. 1), one has been deleted from the program (No. 7) and two are yet to be installed (Nos. 4 and 6). Out-of-bank flooding still occurs on the average of once per year. No formal channelization work has been undertaken but some reaches of the channel have been modified by local landowners and the Indiana Department of Highways. The design specifications for the channelized reach were amended in 1978 to include 13,600 feet of channel shortening, which represents a length reduction of 17.3%.

The use of TR-25 method (SCS, 1977) to design the channelized portion required that the bed and banks of the channel be rip-rapped to prevent channel erosion which would have added about one million dollars to the project cost. The objective of this study, which was conducted in 1983, was to determine whether the channelization could be undertaken without deleterious results if the rip-rap were omitted.

WATERSHED PHYSIOGRAPHY AND GEOLOGY

The watershed has a drainage area of 66.7 square miles, and the approximate length of the channel, which flows in a NW-SE direction, is 15 miles. Average valley width is 4.5 miles and the maximum relief in the watershed is 543 feet. The eastern part of the watershed lies within the Scottsburg Lowland physiographic unit, which is underlain by relatively unresistant New Albany Shale of late Devonian to middle Mississippian age (Schneider, 1966). The western part of the watershed lies within the Norman Uplands physiographic unit. It is underlain by relatively resistant siltstones and shales of the Borden Group, which are of early to middle Mississippian age (Schneider, 1966).

The Pleistocene-Holocene age alluvial-valley fill differs in the upper, middle, and lower part of the valley. In the upper part of the valley, very cohesive Pleistocene age lacustrine sediments are overlain by non-cohesive graveliferous Holocene age sediments. The gravels are overlain by finer grained sediments that were derived from land use changes resulting from European occupation of the area. Moderately cohesive Holocene age lacustrine sediments overlain by gravels and sands are in the middle part of the valley. In the lower reaches of the valley the Holocene age lacustrine sediments are overlain by the historic alluvium and the gravels are absent (Harvey et al., 1984).

Both the New Albany and Borden shales crop out in the bed of Muddy Fork along its entire length. The shales form a paleosurface that has been exhumed by degradation in Muddy Fork within the last 20 years. The outcrops of shale are controlling the local bed slope of the channel at 21 locations.

GEOMORPHIC ANALYSIS

The geomorphic analysis involved an integration of field studies and engineering analysis. It included determining the historical condition of the channel and watershed, quantifying the existing conditions, and predicting the likely response of the channel system to the proposed modifications.

Historical Condition

The purpose of determining the historical condition is to ascertain the equilibrium form of the system prior to man's activities. On the assumption that little modification had occurred to the channel of Muddy Fork, the sinuosity (Po) of the channel was taken to represent the historical condition. Sinuosity, which is the ratio of valley slope to channel slope (Vs/Cs) or the ratio of channel length to valley length (C_L/V_L) was determined by measuring the valley and channel reaches on the watershed map (1:24,000). The channel was divided into 8 reaches (Table 1). The average valley slope for each reach was obtained from the left top-bank profile that was surveyed in 1983. The historical channel slope was obtained by dividing the valley slope by the sinuosity of the reach. The current average channel bed slope for each reach was obtained from the 1983 surveyed longitudinal profile. Except for the reach between Souders Branch and Fordyce Branch (Fig. 1) the current bed slope exceeds the historical slope. Recomputing the sinuosity of each reach (Pc) using the current channel slope showed that, with the exception of the Souders-Fordyce Branch reach, the channel is not in equilibrium because, by definition, the sinuosity of an equilibrium channel cannot be less than unity (Table 1).

Table 1. Valley and channel slope and length data for eight reaches of Muddy Fork.

Reach	Valley Length V_l (ft)	Channel Length C_l (ft)	Sinuosity[1] (P_o)	Valley Slope V_s (ft/ft)	Original Channel Slope[2] C_{so} (ft/ft)	Existing Channel Slope C_{se} (ft/ft)	Recomputed Sinuosity[3] P_c
Packwood Branch to Souders Branch	5200	5680	1.09	0.0050	0.0045	0.0055	0.91
Souders Branch to Fordyce Branch	3240	3720	1.15	0.0037	0.0032	0.0028	1.32
Fordyce Branch to Koetter Branch	6800	7100	1.04	0.0019	0.0018	0.0024	0.79
Koetter Branch to Persimmon Run	12280	14860	1.21	0.0021	0.0017	0.0023	0.91
Persimmon Run to Carwood	6520	7980	1.22	0.0016	0.0013	0.0017	0.94
Carwood to Wilson	10520	16620	1.58	0.00085	0.00054	0.00093	0.91
Wilson to Perry Crossing	16000	29560	1.85	0.00073	0.00039	0.0017	0.43
Perry Crossing to Silver Creek	12000	19740	1.65	0.00079	0.00048	0.00075	1.05

[1] $P_o = C_l/V_l$; [2] $C_{so} = V_s/P_o$; [3] $P_c = V_s/C_{se}$

Existing Condition

If the interpretations of the sinuosity data are correct, then evidence for the non-equilibrium state of the channel should be present. Thirty three valley and channel cross sections had been surveyed along the length of Muddy Fork in 1962. In 1983 these cross sections were resurveyed and another twenty cross sections were surveyed. Comparison of the 1962 and 1983 surveyed cross sections showed that degradation had occurred in the middle and upper reaches of the channel except where uncontrolled tributaries were located. In the lower reaches of the channel, in general, aggradation had occurred (Fig. 2). The cause of the degradation is not known. The presence of shale outcrops in the bed of the channel in the lower reaches in 1962 indicate that the degradation commenced prior to this time. Installation of the five floodwater-retarding structures in the upper and middle parts of the watershed may have aggravated the situation by reducing sediment yield to Muddy Fork. Local extraction of gravel may have compounded the problem.

In a degrading or incised channel, the increase in channel depth predisposes the banks to gravity failures once a critical bank height is attained (Little et al., 1982; Schumm et al., 1984) and, therefore, the observed degradation in Muddy Fork (Fig. 2) should have resulted in bank failure and channel widening. Channel depth at bankfull stage was determined from the surveyed cross sections (1983). Channel depth along Muddy Fork is remarkably constant (mean = 10.4 feet; standard deviation = 2.2 feet) and this is a function of the constant depth of the alluvial-valley fill above the shale paleosurface which now controls channel depth along the length of Muddy Fork. Field inspection showed that the highest frequency of bank failure was occurring in the middle and upper reaches of the channel, upstream of Carwood (Fig. 1). In the lower reaches of the channel very little bank failure was observed. In fact, lateral accretion of fine-grained sediments on the banks was observed. The occurrence of bank failure is, therefore, closely related to the distribution of the channel-forming sediments.

Bed and bank sediment samples were collected at seven locations along the channel of Muddy Fork. The sampling procedure was that determined by Schumm (1961) to describe the sediments that form the wetted perimeter of a channel (Table 2). The data show that median grain size (d_{50}) of the bed sediments decreases in the downstream direction, which is related to the fact that the lower tributaries in the watershed are not delivering gravel to Muddy Fork. The percentage of silt and clay (M) in the perimeter sediments (Schumm, 1961) increases downstream of Carwood (station 567+57), and this correlates with the reduction in frequency of observed bank erosion. The grain size data indicate that with a relatively constant bank height, the incidence of bank failure is a function of the sediments that comprise the banks. Upstream of Carwood the critical bank height has been exceeded for the less cohesive (i.e., lower M values) sediments, whereas downstream of Carwood the more cohesive sediments can support a greater bank height and they are, therefore, stable. Bank failure directly effects channel width and this is shown by the greater channel widths, at bankfull stage, of the reaches upstream of Carwood (mean = 84.2 feet; standard deviation = 27.8 feet) as compared to channel widths in the reaches downstream of Carwood (mean = 66.8 feet; standard deviation = 14.8 feet).

From the preceding discussion of the planform and channel characteristics of Muddy Fork it is evident that the channel of Muddy Fork is currently not in an equilibrium state, but the cohesiveness of the perimeter sediments in the reaches downstream of Carwood is preventing channel adjustment. In other words, in this reach the force-resistance ratio (Richards, 1982) is dominated by the stability of the banks and in effect the channel of Muddy Fork is energy deficient. This can be tested by determining if the morphologic variables of the channel (e.g., width, cross section area) correspond with the discharge, expressed as total stream power (Wolman and Brush, 1961; Henderson, 1961).

The TR-20 computer program (SCS, 1965) was used to derive the hydrology of Muddy Fork. The HEC-2 program (HEC, 1981) was used to generate the in-channel hydraulics. A Manning's "n" value of 0.04 was used in the computations which reflected the high degree of roughness caused by the numerous debris jams in the channel. The 1-year discharge was used in the analysis because this represents the bankfull discharge of the channel. The 1-year discharge for both the existing and improved (i.e., channelized) conditions were computed. The existing condition showed a decrease in discharge in the downstream direction because discharge is computed as a function of cross-sectional conveyance and this is limited by the very high roughness imparted by the debris jams. Also, channel width is greater

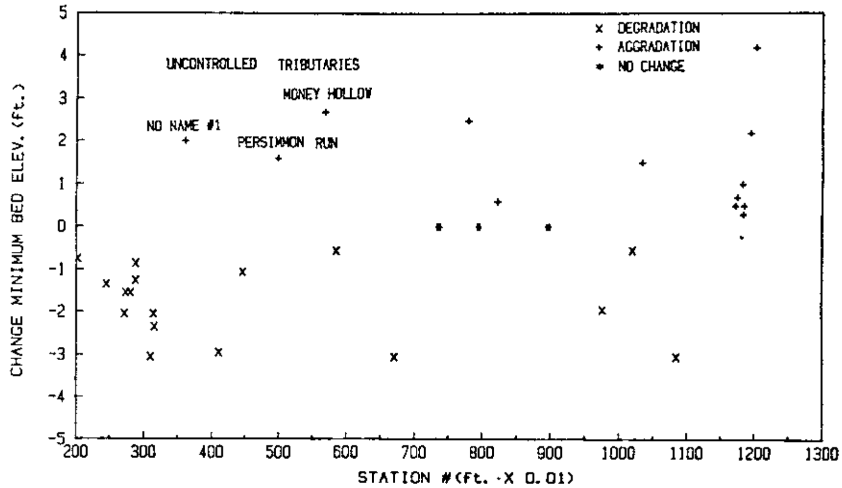

Fig. 2. Change in minimum bed elevation (1962-1983) plotted against station, Muddy Fork.

Table 2. Grain size data for channel bed and banks, Muddy Fork.

Station (ft)	Banks d_{16} (mm)	Banks d_{50} (mm)	Banks d_{84} (mm)	Banks $(d_{84}/d_{16})^{0.5}$	Bed d_{16} (mm)	Bed d_{50} (mm)	Bed d_{84} (mm)	Bed $(d_{84}/d_{16})^{0.5}$	M^1 % silt and clay
330+21	0.0048	0.03	0.068	3.83	0.045	7.6	20.0	21.08	34.8
468+50	0.006	0.034	0.075	3.54	0.70	8.80	20.0	5.35	27.3
491+41	0.011	0.10	9.60	29.54	2.50	10.0	24.0	3.10	17.2
579+35	0.003	0.026	0.052	4.16	0.60	6.0	16.8	5.29	38.9
778+00	0.003	0.027	0.054	4.24	0.05	7.0	24.0	21.90	32.1
794+00	0.003	0.015	0.045	4.24	0.05	6.4	14.8	17.20	38.5
1018+58	0.002	0.013	0.04	4.47	0.03	5.8	14.2	21.76	40.7

1 $M = \frac{(\%\ \text{bank} \times 2d) + (\%\ \text{bed} \times w)}{2d + w}$ (Schumm, 1961)

in the upstream direction. The improved condition assumes that the remaining two floodwater-retarding structures have been emplaced and that the channelization has taken place.

Watson and Harvey (1984) and Harvey and Watson (1985) used a relationship between total stream power, d_{50}, and the channel wetted perimeter derived by Henderson (1961) to formulate a least squares regression equation that related channel top width (T.W.) to total stream power (γ.Q.S), median grain size (d_{50}) of the bed sediments, and the percent silt and clay (M) in the channel perimeter sediments. The relationship had the general form:

$$\text{T.W.} = a\,(\gamma.Q.S)^{b}\,d_{50}^{-c}\,M^{-d} \tag{1}$$

The channel of Muddy Fork was subdivided into two reaches, upper and lower, on the basis of the observed differences in bank materials and the frequency of bank failure sites. The upper reach is located upstream of Carwood and the lower reach is that between Carwood and the confluence with Silver Creek (Fig. 1). The Muddy Fork data for the two reaches were subjected to the same analysis (Eq. 1). The least squares regression relationship for the upper reach is:

$$\text{T.W.} = 2337\,(\gamma.Q.S)^{0.08}\,d_{50}^{-0.22}\,M^{-1.01} \qquad (r^2 = 0.48) \tag{2}$$

and that for the lower reach is:

$$\text{T.W.} = 1451\,(\gamma.Q.S)^{-0.15}\,d_{50}^{-1.28}\,M^{-1.91} \qquad (r^2 = 0.88) \tag{3}$$

The coefficient of determination (r^2) value for the upper reach (Eq. 2) indicates a rather weak relationship, but it does show that the relationship conforms to those of Henderson (1961), Watson and Harvey (1984) and Harvey and Watson (1985). The relatively poor fit of the data results from the active channel widening as a function of bank failure and control of the channel slope by outcrops of shale.

In contrast to the upper reach, the lower reach relationship (Eq. 3) shows a good fit to the data ($r^2 = 0.88$), but the form of the relationship is contrary to that of the previous studies. Top width is related negatively to all three independent variables. This is due to the reduction of both discharge and slope in the downstream direction. The strength of the relationship and the negative correlation with the independent variables further confirm the non-equilibrium status of the lower reach of Muddy Fork, even though there is little field evidence of channel adjustment.

PROPOSED MODIFICATION

The morphologic, grain size and energy relations for Muddy Fork (Eq. 2 and 3) indicate that the channel is not in a state of equilibrium but there appears to be a deficiency of energy in the lower reach of the channel with respect to the resistance of the perimeter sediments. If this argument is valid, then it also follows that the existing condition of the channel provides a basis for testing the proposed channelization design (stations 417+00 to 1202+70, Fig. 1).

The existing top width and design top width for the channelized portion of the channel were plotted against station (Fig. 3). Also plotted on the figure are adjusted top widths. These were derived from Eq. 2 and 3 and they represent the predicted top widths when the 1978 design stream power is used in the existing condition relationships. Figure 4 shows that in general the adjusted top width is about 15% narrower than the design top width. This suggests that the proposed design is within the error estimate of the predicted top widths for the existing condition, and therefore, it is unlikely that channel erosion (i.e., widening) will occur as a result of channelization since little channel erosion is observed under existing conditions.

To test this, a limited study of Elk Creek, which had been channelized in 1962 prior to the use of TR-25 design methodology, was conducted. Elk Creek is also located in the Scottsburg Lowland physiographic unit and it has a similar drainage area and topography. Samples of the bed and bank sediments were collected at four locations (Table 3). The median grain size (d_{50}) of the bed sediments of Elk Creek is within the range of those encountered in Muddy Fork (Table 2) and the d_{50} of the bank materials is also similar.

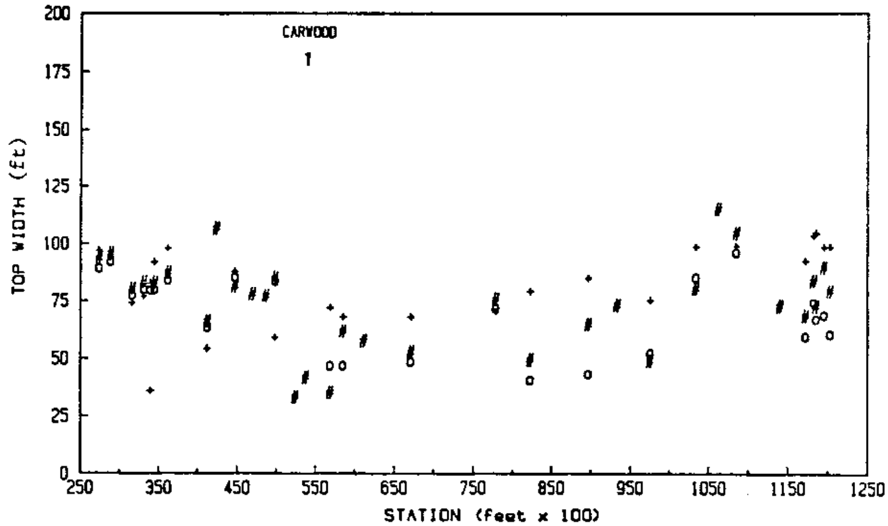

Fig. 3. Channel top width plotted against station, Muddy Fork: + - design; # - existing; O - adjusted.

However, the M values in Elk Creek are considerably lower than those computed for the lower reach of Muddy Fork which indicates that the perimeter sediments of Elk Creek are less cohesive. Channel width and depth measurements were made where the sediment samples were obtained, and they were compared with the design specifications for the channel. Except for one reach these channel widths were substantially the same as the design widths which was supported by the general lack of observed bank erosion. Between stations 877+10 and 857+08 severe bank erosion was observed. Comparison of pre-and-post channelizations lengths indicated that this reach had been shortened by about 36% and it was also located in a relatively steep reach of the valley (0.0018 ft/ft). Channelization of Elk Creek, which involved channel shortening and increased channel slopes in materials that were less cohesive (Table 3) than those of Muddy Fork, showed no adverse effects except where the length was reduced by 36% and the slope was increased by 54.5%.

To test the hypothesis that both Elk Creek and Muddy Fork are energy deficient with respect to the resistance of the perimeter sediments a tractive stress analysis was conducted for the present channel of Elk Creek. The mean value of the ratio of actual tractive stress (τ) to the allowable tractive stress (τ_a) generated by the 1-year discharge for the "as-built" condition ($0.029<n<0.031$) is

2.16. The ratios (T/T_a) were also calculated for "n" values of 0.025 and 0.40 and they are 3.48 and 1.47 respectively (Table 4). The lack of observed channel adjustment following channelization of Elk Creek indicates that the allowable tractive stress is too conservative for these perimeter sediments.

Table 3. Grain-size data for channel bed and banks, Elk Creek.

Station (ft) (Bridge Number)	Banks				Bed				M^1 % silt and clay
	d_{16} (mm)	d_{50} (mm)	d_{84} (mm)	$(d_{84}/d_{16})^{0.5}$	d_{16} (mm)	d_{50} (mm)	d_{84} (mm)	$(d_{84}/d_{16})^{0.5}$	
949+10 (1)	0.013	0.035	0.43	5.75	0.12	0.60	3.30	5.24	26.46
866+77 (2)	0.016	0.065	0.37	4.81	0.09	0.43	4.50	7.07	25.40
776+48 (3)	0.047	0.083	0.41	2.95	2.50	9.60	24.0	3.10	9.00
742+06 (4)	0.014	0.037	0.25	4.23	2.00	6.50	15.0	2.74	17.00

[1] $M = \frac{(\%\ \text{bank} \times 2d) + (\%\ \text{bed} \times w)}{2d + w}$ (Schumm, 1961)

Since the perimeter sediments of Muddy Fork are more cohesive than those of Elk Creek, then a comparison of T/T_a ratios derived for Muddy Fork with the mean value for Elk Creek provides a means of testing the contention that channelization will not result in channel erosion in Muddy Fork (Fig. 3). Manning's "n" values of 0.025 and 0.04 were used in the computations to represent "as-built" and "aged" conditions for the entire channel, the lower reach and the upper reach (Table 4). The mean value of T/T_a values for Elk Creek is about 1.3 times that of Muddy fork with an "n" value of 0.025 but the T/T_a values are about equal for an "n" value of 0.04. However, when Muddy Fork is divided into the upper and lower reaches it is apparent that the two reaches will behave differently. When "n" values of 0.025 and 0.04 are used for the lower reach, values of T/T_a are 2.1 and 1.7 times, respectively, less than the mean value for Elk Creek. This suggests that the proposed channelization of the lower reach of Muddy Fork is unlikely to result in significant channel erosion even if a large event occurs prior to the establishment of an aged condition. In contrast, the T/T_a values for the upper reach of Muddy Fork are 1.3 and 1.8 times greater than the mean Elk Creek value when "n" values of 0.025 and 0.04 are used. This suggests that channel widening could occur in the channelized portion of the upper reach. Since the bed of the channel is controlled by outcrops of shale, further degradation is unlikely and, therefore, channel widening could be prevented by the emplacement of suitably sized rip-rap (Harvey et al., 1983).

Table 4. Computed mean values of τ/τ_a for Elk Creek and Muddy Fork.

Channel	As-Built Condition (0.029 < n < 0.031)	n = 0.025	n = 0.040
Elk Creek	2.16	3.48	1.47
Muddy Fork			
273+28-1201+70 (entire channel)	--	2.76	1.55
584+75-1201+70 (lower reach)	--	1.62	0.87
273+28-584+75 (upper reach)	--	4.57	2.63

SUMMARY AND CONCLUSIONS

A geomorphic investigation that involved field studies and engineering analysis was conducted to determine whether the proposed channelization of Muddy Fork would result in channel erosion if the channel were not armored with rip-rap. Omission of the rip-rap would result in a cost savings of about 1 million dollars thereby improving the benefit-cost ratio for the project significantly.

Analysis of planform (Table 1) and morphometric characteristics of Muddy Fork indicated that the channel was not in a state of equilibrium. However, the expression of the non-equilibrium status of the channel was closely related to the cohesiveness of the perimeter sediments in that significant bank failures were only observed in the reach upstream of Carwood, where the least cohesive sediments were present (Table 2). This was confirmed by two multivariate relationships between channel top width, stream power, d_{50}, and M (Eq. 2 and 3). fitting of the proposed channelization design to these relationships (Fig. 3) indicated that erosion of the channel was unlikely to occur. This was supported in the lower reach, by a comparison of ratios of actual to allowable tractive stress (τ/τ_a) between Elk Creek, which had not adjusted as a result of channelization in 1962, and the lower reach of Muddy Fork (Table 4). The upstream portion of the proposed channelization is located within the upper reach of Muddy Fork and the tractive stress comparison with Elk Creek indicates that channel bank erosion could eventuate (Table 4). This can be prevented by emplacing suitable sized rip-rap on the banks since further channel degradation is unlikely to occur.

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SIMILARITY OF BANK PROBLEMS ON DISSIMILAR STREAMS

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ABSTRACT

The objective of this paper is to describe the similarities and dissimilarities in channel-bank adjustments for several channels which occupy distinctly different physiographic positions. Problem reaches of the Black River, Louisiana, and several small tributaries in northern Mississippi all have excessive depths. The northern Mississippi channels have entrenched due to relatively recent knickpoint incision whereas the depth of the Black River is relic. For all of these channels, the dominant form of bank instability is mass failure, particularly tension crack accelerated slab failure. Failure frequencies for these two channels, however, are expected to be considerably diverse. Worst case failure conditions due to bank saturation are controlled by local climatic conditions for the northern Mississippi creeks. However, high stages on the Black River caused by backwater from the Red, Atchafalaya, and Lower Mississippi Rivers control the development of worst case conditions. An additional difference between these two channels is the occurrence of deep-seated, slip circle failures on the Black River. Although the lower Ouachita River also has excessive channel depths, the banks are generally stable due to stage control by a series of locks and dams. A generalized stabilization strategy is proposed.

INTRODUCTION

Stream-bank instability is a serious problem for many tributary channels in the Lower Mississippi River Basin. This basin includes a wide variety of natural and man-induced conditions that potentially could effect diverse channel-bank adjustments, thereby directly and indirectly affecting bank stability-instability relations. The purpose of this paper is to compare bank instability problems for two dissimilar study areas: the Black-Ouachita River system in Louisiana and Arkansas; and Goodwin, Hotophia, Johnson, and Long Creeks in northern Mississippi.

STUDY AREAS

The Black-Ouachita River System (Fig. 1).

The watershed of the Black-Ouachita River system is classified as a subdrainage area of the Red River Drainage Area of the Lower Mississippi River Basin (Soil Conservation Service, 1963). This is a large watershed, 64,000 km^2 (24,700 square miles) in size. It is relatively complex,containing five distinctly different land resource regions in three separate physiographic sections; the Ouachita Mountains Physiographic Section of the Interior Highlands and the West Gulf Coastal and Mississippi Alluvial Plains of the Atlantic Plain. Major tributaries include Moro Creek and the Saline, Caddo, Little Missouri, Boeuf and Tensas Rivers. The Black River is formed by confluence of the Tensas and Ouachita Rivers at Jonesville, Louisiana.

For the natural system, the numerous swamps, lakes, and bayous downstream of the Mountain Section had a larger storage capacity to buffer discharges for storm events and hence to minimize coarse sediment transport through the system. Data presented by Markham (p.27, 1935) for an April 1927 storm illustrate this buffering. He stated, "The maximum discharge for the 1927 flood is estimated to have been 134,000 second-feet near Hot Springs from an area of 1,400 square miles, which would be 95.7 cubic feet per second per square mile. At Camden with a drainage area of 5,330 square miles, it is estimated to have been 145,000 second-feet, or at the rate of 27.2 cubic feet per second per square mile. At Monroe with a drainage area of 16,820 square miles the maximum discharge was 77,560 second-feet, which would give a run-off of 4.6 cubic feet per second per square mile." Peak velocities also decrease downstream and are less than 1 m/s for the Black River. Backwater influences from the Mississippi River via the Red River also contributed to these relatively low flow energies for the Black River. Fisk (1938) stated that these backwater levels existed to an elevation of about 12 m msl.

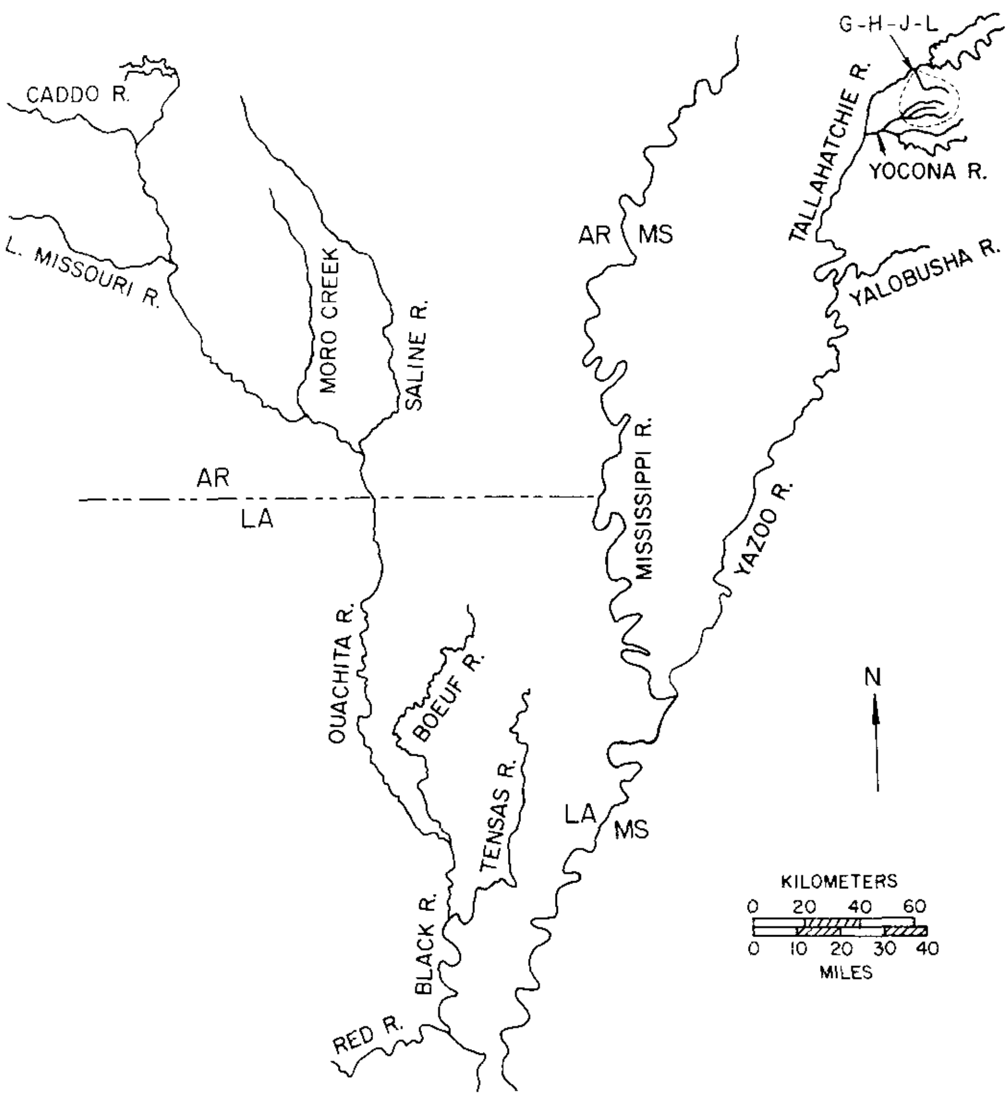

Fig. 1. Location map for the Black-Ouachita River system, Arkansas and Louisiana and for the Goodwin-Hotophia-Johnson-Long Creeks, northern Mississippi (noted as G-H-J-L) study areas.

Undoubtedly, the channels were adjusting during pre-and early-settlement times, but the rate of change was exceedingly slow due to the natural storage of flood flows in combination with high backwater levels which together inhibited the transport of coarse sediment. Evidence for the general stability of this system has been presented by Biedenharn et al. (1983) who reported effectively no change in the plan-form geometry of this system over the past 160 years. Additional but unpublished data show no systematic change in cross-section geometry of this system over the past 86 years.

Relatively recent flood abatement activities, however, have changed the flow regime, primarily by increasing the within-channel flow duration. Reservoirs constructed in the mountain section on the Caddo, Little Missouri and Ouachita Rivers decreased peak flows from these source areas but increased flow duration. Overbank storage and/or flood flows were further reduced and within-channel flows were increased by the construction of levees and tributary-channel plugs. Concurrently, management activities on the Lower Mississippi River reduced backwater influences on the Black and Ouachita Rivers (via the lower Old and Red Rivers), resulting in increased rates of changes in stage. The magnitude of this influence is not known. The construction of the several locks and dams has minimized rapid stage changes upstream of Jonesville Lock and Dam but stage changes downstream are both rapid and of large magnitude. Figure 2 illustrates this condition. It should be noted that the stage differences between Lower Jonesville and Acme are independent of absolute stage, implying backwater control by the Red River.

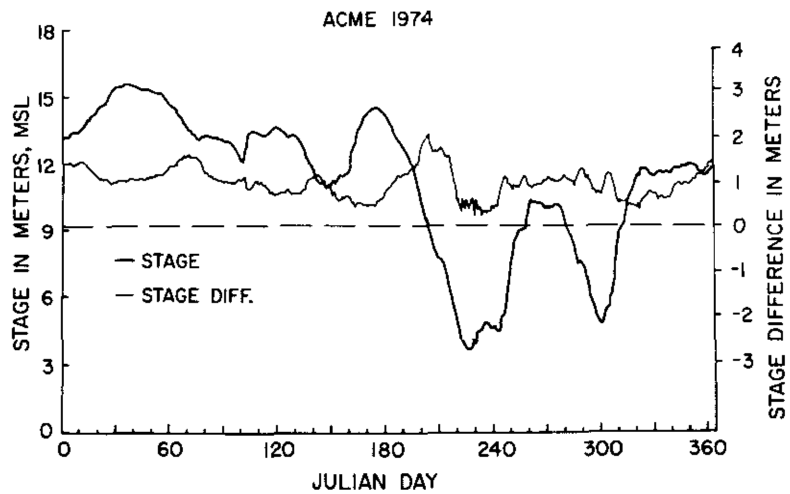

Fig. 2. Black River stage at Acme, LA (heavy line) and stage difference (light line) between lower Jonesville Lock and Dam (at river mile 25) and Acme (at river mile 0) for 1974.

Goodwin, Hotophia, Johnson, and Long Creeks.

These four creeks, identified as G-H-J-L on Fig. 1, are all located in the southeastern quarter of Panola County, northern Mississippi. All are tributaries to the Yazoo River, but are too small to be formally classified as subdrainage areas of the Lower Mississippi River Basin. Drainage areas are

21.4 km^2 (8.26 square miles) for Goodwin Creek, 53.6 km^2 (20.7 square miles) for Johnson Creek, 73.0 km^2 (28.2 square miles) for Long Creek, and 94.2 km^2 (36.4 square miles) for Hotophia Creek. These four watershed systems are relatively simple, in comparison to the Ouachita-Black River system. All four watersheds are within the East Gulf Coastal Plain Physiographic Section of the Atlantic Plain, all are in the same land resource region, and all have a loess veneer overlying unconsolidated sands to gravels in the uplands. Additionally, all four channels conflow with the trunk stream immediately downstream of major flood-prevention reservoirs. Flows are flashy following storm events, with peak flow velocities approaching 2.5 m/s for some reaches. One minor difference between these four watersheds is that Hotophia Creek is a tributary of the Tallahatchie River whereas Goodwin, Johnson and Long Creeks conflow to form Peters Creek, a tributary of the Yocona River. The Yocona River, in turn, conflows with the Tallahatchie River in the Mississippi Alluvial Plain.

The nature of the valley-fill deposits that crop out as channel bed and bank materials in this area have been previously documented. As summarized by Grissinger and Murphey (1984) the most common valley-fill units include:

1. Early-Holocene depositional sequence (ca. 9000 to 13,000 years Before Present). The most prevalent lithologic unit in this sequence is the massive silt, a fine-textured, dense unit which frequently exceeds 4 m in thickness. It has a distinctive polygonal structure which creates planes of weakness and the prevalent type of bank instability is gravity-induced block failure along these weakness planes. The massive silt unit overlies an older member composed of three lithofacies; the channel lag, bog-type or gray silt units. These units contain abundant wood and other organic detritus and the ages of carbon samples from these phases are early-Holocene. The gray silt material is cohesive and relatively more resistant to scour than either the bog-type or channel lag materials.

2. Mid-Holocene depositional sequence (ca. 5000 to 6500 years Before Present). The channel fill lithologic unit is the only member of this sequence. It is a relatively coarse-textured material reflecting a high energy fluvial system. This unit is a minor constituent of the valley-fill complex but is frequently associated with intense but small-sized bank failures. Such failures result from the lack of cohesion in this material.

3. Late-Holocene depositional sequence (less than ca. 2500 years Before Present). This sequence includes the postsettlement and meander-belt units. Both units include vertical accretion deposits over lateral accretion deposits and both units are relatively unconsolidated. Bank failure is usually preceded by the development of tension cracks parallel to the channel bank. These materials erode readily.

These property variations between individual units together with channel deepening and channel realignment in their valleys have produced relatively complex channel adjustments in this study area. Channel realignments have exposed a number of valley-fill units as channel bed and bank materials that would not otherwise be exposed. For the most part, these realignments were imposed prior to 1937 (the date of the earliest aerial photography). Since 1937, channel entrenchment has become progressively more significant, primarily by knickpoint-type incision, and the resultant increased bank height has greatly increased the frequency of mass failure due to gravity. For a given bank, the probability of mass failure depends upon the strength properties of the individual valley-fill units that form the bank.

BANK STABILITY

Field Observations.

Repeated reconnaissance surveys have been made in the northern Mississippi four-channel study area since 1977 to identify bank failure mechanisms and to develop associations between failure mechanisms and the various lithologic units. For these surveys, the lithologic units were used as surrogates for general bank material properties. These surveys have established (1) that the dominant bank failure mechanism is slab failure accentuated by tension crack development, and (2) that this type of failure occurs primarily on banks composed largely of late-Holocene materials. The tension cracks commonly observed were vertical separations, generally parallel with the bank, and 0.6 to 1.2 m from the bank edge. A less frequently observed failure mechanism is undercutting of the early-Holocene massive silt by erosion of either channel lag or bog-type materials in a bank toe position, leading to disarticulation of massive silt blocks along planes of weakness induced by the polygonal structure. This type of lithologically-controlled failure has resulted in excessive channel widening and/or bendway development at locations where the banks are made up of late-Holocene units relative to locations where the banks are early-Holocene lithologic units (Grissinger and Murphey; 1983, 1984). Although failure frequencies have not been documented, average annual bank erosion rates of 3.8 metric tons per meter of bank length have been documented for a 3.86-km length of Goodwin Creek (Murphey and Grissinger, 1985).

Aerial and ground reconnaissance surveys of bank instability problems on the Black-Ouachita Rivers were conducted in 1983. Bank problem sites were identified and failure mechanisms evaluated using criteria described by Little and Grissinger (1985). Most of the problem areas identified during this inspection period were located on the outside bank of bendways and an unusually high percentage occurred on the Black River downstream of Jonesville Lock and Dam where flow was uncontrolled. Mass failures were the dominant processes affecting bank stability, and were most frequently the tension crack accentuated slab type. Circular slip failures were less frequently observed.

Bank Failure Analysis.

The most frequently observed failure mechanism for both study areas was tension crack accentuated slab failure. This type of instability has been analyzed by Little et al. (1982) for banks in the northern Mississippi four channel study area, using the limit analysis for a logarithmic spiral toe failure. The general form of the stability equation used in this analysis was

$$\frac{\gamma}{c}\,(H_c' + y) = N_s = f(\phi,\ i) \qquad (1)$$

H_c' = critical height of bank with tension crack
y = depth of tension crack
γ = bulk unit weight
c = cohesion
ϕ = friction angle
i = bank angle
N_s = dimensionless stability number

Values for N_s used in this analysis were taken from Chen (1975). Assumptions were zero tensile strength and tension crack depth equal to one-half the critical bank height. These assumptions have been discussed by Little et al. (1982).

Results of this bank stability analysis for Hotophia Creek are shown in Fig. 3 (after Little et al., 1982, Fig. 7). Worst case conditions, that is minimum strength and maximum bulk unit weight at or near saturation, with tension cracks were found to accurately discriminate between banks that were stable and those that failed during a study period of about 1.5 years. Qualitative agreement was also noted for three other reaches on Goodwin and Johnson Creeks for banks composed of differing lithologic units and hence differing strengths.

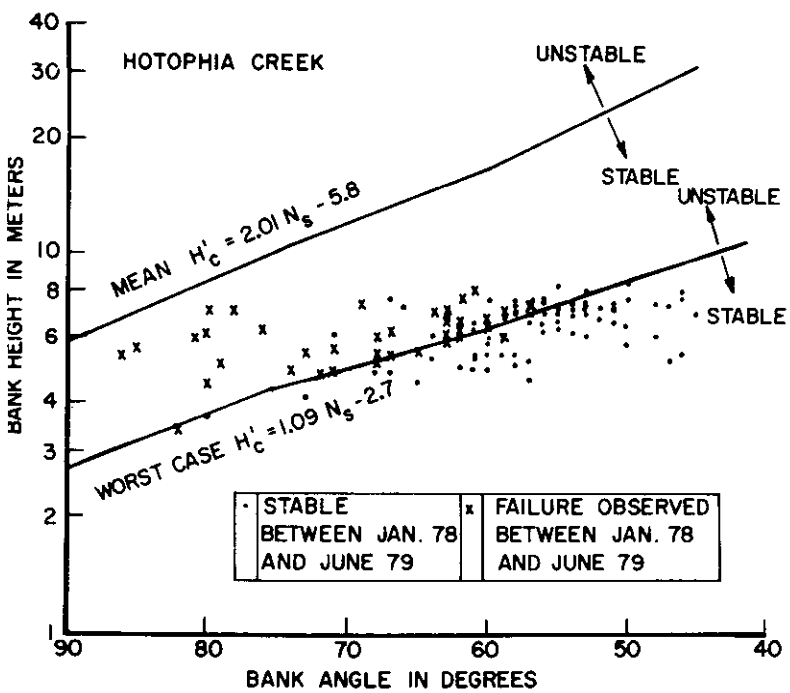

Fig. 3. Bank stability graph for Hotophia Creek, MS (after Little et al., 1982). Mean and worst case critical heights (H_c') were calculated using equation (1) in the form $H_c' = N_s\ (c/\gamma) - y$.

The same stability analysis was used to analyze banks on the Black River downstream of Jonesville Lock and Dam, using cohesion and friction angle data published by the U. S. Army Corps of Engineers (1968, 1976) for nearby areas.

All materials are late-Holocene Mississippi River deposits. The frequency distribution for 301 values of cohesion is presented in Fig. 4. All values are assumed to be worst case conditons and the peak value of 36 KPa was used in the analysis. The majority of the samples had friction angles near zero, therefore for this analysis, a 0° friction angle was used as a conservative estimate. For bank angles greater than 20°, the critical bank heights with and without tension cracks (Fig. 5) were less than the observed bank height of 14 m or greater. Although site specific comparisons could not be made due to the nature of the strength values (Fig. 4), the calculated stability analysis was in general agreement with the field observations. Banks had failed and these failures were concentrated (1) along the outside bank of bendways where the bank angles were greatest and (2) downstream of Jonesville Lock and Dam where stage changes were greatest.

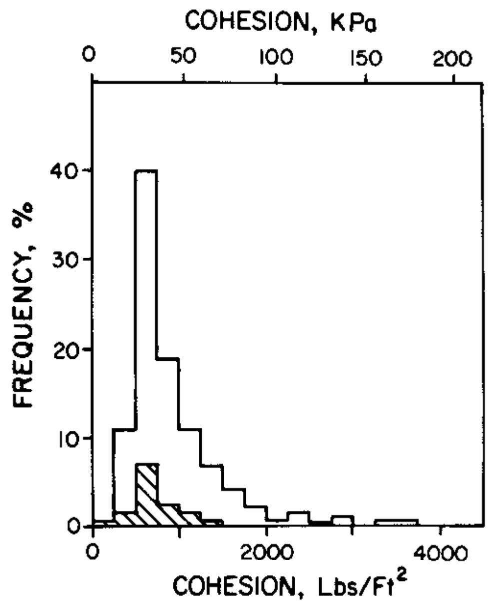

Fig. 4. Cohesion frequency for 301 samples from Cocodrie and Wild Cow Bayous, LA. The shaded insert is the frequency for the 59 samples from Cocodrie Bayou, expressed as percentages of the total sample. (Source: U. S. Army Corps of Engineers, 1968 and 1976.)

This analysis addresses the problem of bank failure for worst case conditions. It does not address the allied problem of failure frequency. Obviously, the failure frequency will be highly site-specific, varying with prolonged wet periods leading to worst case (material strength) conditions, bank oversteepening and/or bed degradation, and the development of tension cracks. Qualitatively, these controls indicate varying frequencies for the two study areas. For the northern Mississippi four channel area, worst case conditions are ultimately controlled by seasonal precipitation patterns whereas for the Black River the worst case conditions are controlled by both seasonal precipitation patterns and backwater conditions from the Red River. Also, due to the low flow energies for the Black River, bank steepening to critical conditions should be relatively slow and should be concentrated along the outside banks of bendways, as observed. In contrast, failures for the northern Mississippi study area should be more frequent due to more rapid bank oversteepening and/or bed overdeepening for this higher energy flow system. Again, these results are in general agreement with the field observations. At this time, however, the influences of these controls cannot be quantified due to insufficient data describing material characteristics and due to inadequate definition of the processes of tension crack development.

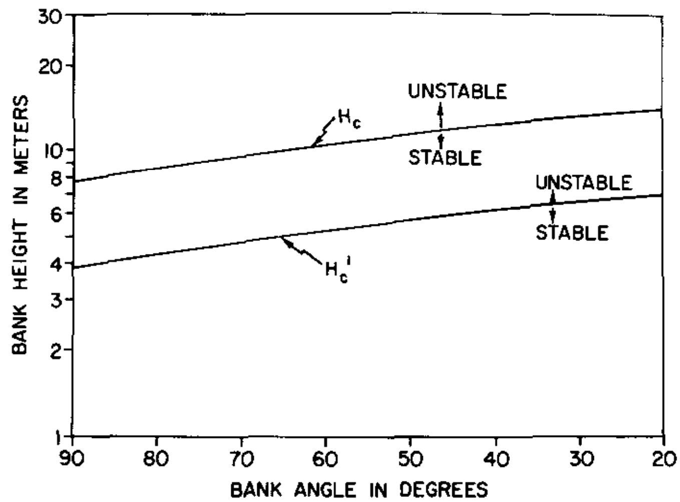

Fig. 5. Bank stability graph for Black River, LA. Critical bank heights with (H_c') and without (H_c) tension cracks were calculated using equation (1) in the form $H_c' = N_s\ (c/\gamma) - y$ or $H_c = N_s\ (c/\gamma)$.

Although circular slip failures and bank disarticulation (due to undercutting) are mechanisms different from tension crack induced slab failures, the controls are generally comparable. The main difference in these controls is that tension crack development is less significant for the circular slip failures and for bank disarticulation. Observations show that circular slip failures occur very slowly (many months), generally at locations where additional bank weight has been added such as by a top bank levee or by continuous saturation from surface ponding. These two mechanisms have not been further analyzed.

Stabilization Strategies.

Bank instability due to mass failure will be a chronic problem for all locations where the flow conditions are sufficient to erode bank slough material from the bank toe. For both study areas, the slough material is fine-textured and relatively easily eroded, resulting in repeated cycles of slough erosion, bank oversteepening, and mass failure. All other conditions being the same, this type of channel widening will continue until the flow is no longer able to erode the slough.

The critical conditions for this type of bank instability suggest two possible stabilization strategies; either flow regulation sufficient to create non-eroding conditions at the bank toe, or bank toe protection. For these two study areas and for comparable areas where the bank slough material is highly erodible, flow regulation is decidedly less feasible than bank toe protection. Although data are not sufficient at this time to adequately develop design criteria for optimum bank toe protection, we believe that criteria should be based on the dual utilization of revetment to protect the bank toe against discrete particle scour and to add sufficient mass such that the effective bank height is less than the actual height by an amount equal to the revetment height. For slab failure with tension cracks, this approach involves:

1. Determine the height of revetment needed to prevent bank scour to an elevation equal to a design percentage of the stage frequency relation.

2. Compute the limiting bank angle-height relations for stability at worst case conditions with tension cracks.

3. Compute the effective bank height as the difference between the actual height and the revetment height, and determine the limiting stable bank angles for this effective height using the preceding stability analysis.

4. For site-specific applications, alternate combinations of revetment height (and hence effective bank height) and bank angle (and hence top-bank channel width) can be calculated to evaluate alternate revetment costs in relation to potential damages to near-channel facilities. An appropriate safety factor would be similarly dependent upon the site-specific conditions.

In essence, this approach is designed to prevent bank toe erosion and to maintain the banks in stable configurations based on a stability analysis for worst case conditions with tension cracks. A comparable approach based on a stability analysis of worst case conditions but without tension crack development would be decidely advantageous. As shown in Fig. 5, the critical bank height without tension cracks (H_c) is about twice that with tension cracks (H_c'). Unfortunately, this aspect of stress release is poorly understood at present. Due to the potential benefits, this subject should be thoroughly evaluated particularly in relation to vegetative cover and to bank angle constraints on tension crack formation. Obviously, this approach presupposes a stable thalweg. If thalweg lowering is anticipated, bed-control structures will have to be incorporated into the stabilization strategy.

SUMMARY

Tension crack accentuated slab failure has been identified as the dominant bank failure mechanism for two distinctly different study areas. One of these areas, the Black-Ouachita River system in Louisiana and Arkansas, is a large complex watershed with readily identifiable relic channel features. Flow energies are low in the downstream part of this system and the banks have been generally stable. Current bank instability is concentrated downstream of Jonesville Lock and Dam because large fluctuations in stage caused by backwater from the Red River significantly influence the development of worst case conditons for mass stability. The other study area, in northern Mississippi, includes four small, relatively simple watersheds. Flow energies are relatively large and the channel has been degraded during recent times by knickpoint entrenchment.

For both areas, a limit analysis for a logarithmic spiral toe failure is in agreement with field observations. Based on this analysis, a general stabilization strategy is presented.

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SEDIMENT STUDIES IN GRAND CANYON, ARIZONA

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ABSTRACT

The effect of impoundment and flow regulation at Glen Canyon Dam on sediment transport and storage in the Colorado River below the dam is currently being investigated by a combination of analysis of historical data and collection and analysis of new data. Suspended sediment, bed material, and bedload are being sampled at three temporary and two long-term gaging stations along the main stem of the Colorado River and in three of its largest tributaries in the study reach. Side-scan sonar and seismic-reflection surveys provide data on bed materials and channel geometry between gages. A sediment-transport model, calibrated with data from the study period, will be used to simulate sediment transport and storage for alternative flow-release patterns. Long-term response of the channel to changes in flow conditions and to tributary floods will be determined by analysis of records from the two long-term gaging stations. Alluvial sand bars in the study reach are ecologically and aesthetically important elements of the river corridor. Long-term response of sand bars to flow conditions will be determined through analysis of historical photographs and bar surveys. Short-term responses of bars to fluctuating flows will be determined by repeated topographic mapping and measurement of selected physical and hydraulic variables. Results of main stem, tributary, and bar studies will be integrated to evaluate the effect of fluctuating flows on sediment transport and storage in the study reach.

INTRODUCTION

The Colorado River between Glen Canyon Dam and Lake Mead is currently the focus of a large-scale environmental study funded and coordinated by the U.S. Bureau of Reclamation, in cooperation with the National Park Service and the U.S. Fish and Wildlife Service. Those agencies, along with the U.S. Geological Survey, Arizona Game and Fish Department, and other agencies and consultants began collection of data along the 387-kilometer (km) river corridor in June 1983; data collection will continue until about February 1986. Final reports for the study are scheduled for completion by April 1987. The purpose of the study is to evaluate, insofar as possible, the influence of regulated releases at Glen Canyon Dam on the terrestrial and aquatic flora and fauna of the Colorado River corridor, on recreational use of the river, and on sediment transport and associated erosion and deposition of alluvium, particularly in the sand-size range. Data collection and analysis are designed to permit participants to study the effect of the present flow-release schedule on the environment in the Grand Canyon, as well as to investigate the probable impact of operationally feasible alternative flow-release patterns. Analysis of long-term data on flow, sediment transport, and section geometry at gaging stations and on changes in channel banks and sand bars between gages will permit investigation of the effects of patterns of predam flow. Collection of short-term data at gaging stations and near bars will permit investigation of the effect of flow-release patterns typical of powerplant releases.

The purpose of this report is to summarize briefly the approach being used to investigate the influence of impoundment and flow regulation on sediment along the Colorado River in Grand Canyon National Park (Fig. 1). Activities of the sediment subteam of the Glen Canyon Environmental Study will be presented, and three other papers included in these proceedings will be introduced. The three other papers deal in greater detail with channel-bottom surveys, sediment-rating curves for the main channel and the tributaries, and a sediment-transport model used to investigate flow-regulation alternatives.

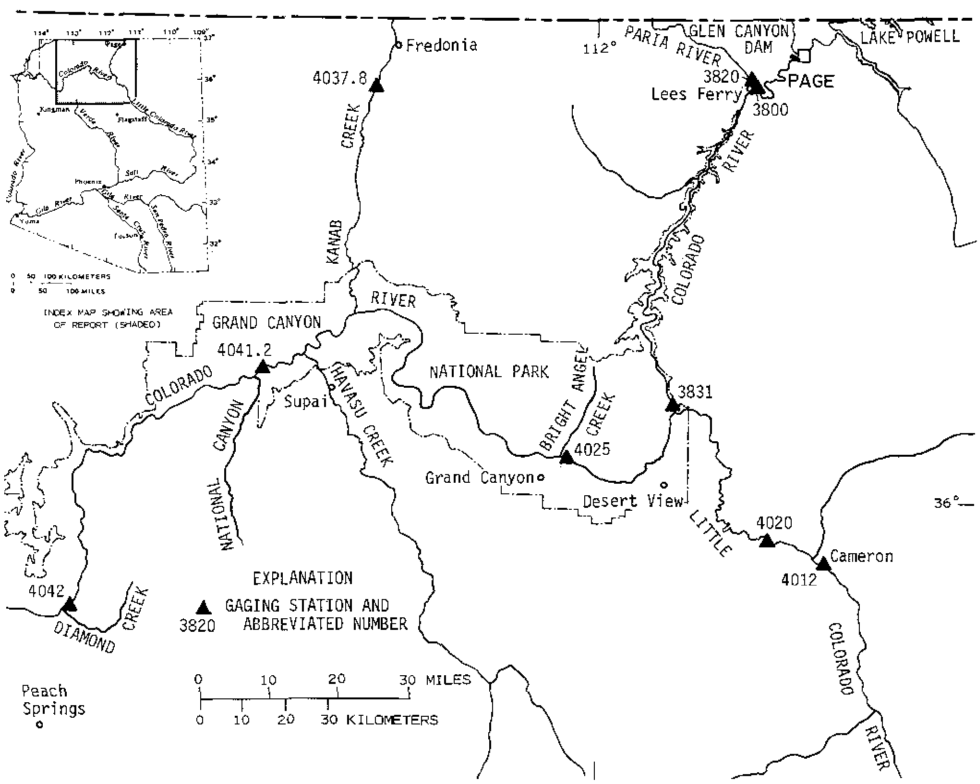

Fig. 1. Location of sampling sites in the study area.

BACKGROUND

Flow regulation at Glen Canyon Dam, completed in 1963, has reduced the mean annual discharge of the Colorado River at Lees Ferry from about 500 to about 350 cubic meters per second (m^3/s) and the annual peak discharge from about 2,600 m^3/s to about 800 m^3/s. Flow through the powerplant is between about 30 and 890 m^3/s. Daily flow releases at Glen Canyon Dam are varied in direct response to power demand. Regulation produces large daily fluctuations (Fig. 2), the magnitude and timing of which vary with day of the week and with season of the year. Virtually all sediment from upstream of the dam is trapped in Lake Powell, which is formed by Glen Canyon Dam. Clear-water releases from the dam and the relatively steep gradient of the river (0.0015 meter per meter) have caused concerns that alluvial material, especially the sand-sized fraction, will eventually be removed from along the Colorado River in Grand Canyon National Park.

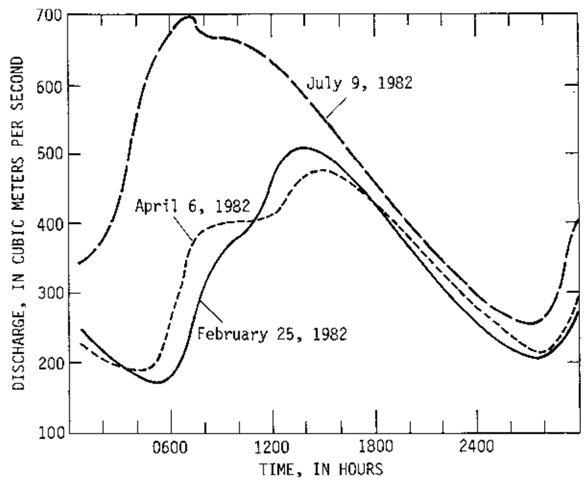

Fig. 2. Typical regulated-flow hydrographs at the U.S. Geological Survey gaging station on the Colorado River near Grand Canyon.

The Geological Survey has operated gaging stations on the Colorado River at Lees Ferry (station number 09380000), 24 km below the dam, and near Grand Canyon (09402500), 165 km below the dam, since 1895 and 1922, respectively (Fig. 1). Suspended-sediment samples were collected intermittently between 1928 and 1965 at the Lees Ferry gage and between 1925 and 1975 at the gage near Grand Canyon. One goal of the sediment phase of the present study is to determine whether long reaches of the river are currently aggrading or degrading. In order to meet this goal, data are being collected at the existing gages at Lees Ferry and near Grand Canyon and at additional locations along the river.

Although the study was planned to investigate the pattern of flow releases that had been in existence for much of the more than 20 years of dam operation, an exceptional combination of weather conditions and management decisions in the Colorado River basin caused record releases from Lake Powell during the summer of 1983. Prior to that time, discharge had exceeded powerplant capacity of 890 m^3/s only in May 1965 and June 1980. Peak discharge was 2,750 m^3/s at the Lees Ferry gage on June 29, 1983. Discharge decreased from that peak to about 730 m^3/s by about August 12, 1983. Discharge remained relatively steady and at or above powerplant maximum releases until March 1985. At that time, the level of Lake Powell had declined and inflows to the lake decreased enough to permit some fluctuation of releases in response to power demand. These unusual flow conditions occurred just as the environmental study was getting underway and necessitated considerable revision of initial project schedules and plans. As of April 1985, data collection was about 50 percent complete. Data collection for fluctuating discharges typical of powerplant releases is planned for fall and winter of 1985-86.

DATA COLLECTION

Data Collection at Gaging Stations

Data are being collected at five sections along the main stem of the Colorado River—the gages at Lees Ferry and near Grand Canyon and three gaging stations established for the period of study (Fig. 1). The temporary stations are Colorado River above Little Colorado River near Desert View (09383100), Colorado River above National Canyon near Supai (09404120), and Colorado River above Diamond Creek near Peach Springs (09404200). The five main stem gages divide the river corridor into four reaches that are 98, 43, 126, and 96 km long, in downstream order.

A cableway is used for discharge measurement and sediment-sample collection at each gage. Stage is recorded at 30-minute intervals at the long-term gages and at 60-minute intervals at the temporary gages. Sediment sampling began June 27, 1983, at the gages at Lees Ferry and near Grand Canyon. The unusually high discharges restricted access to the temporary gaging stations and the last station, above Diamond Creek, was not activated until August 5, 1983. Field parties of two persons were stationed at each gage during the 1983 sampling period. Personnel were rotated and camps supplied every 10 days by motorized raft. Sampling continued until December 13 at Lees Ferry and above the Little Colorado River, December 14 near Grand Canyon, and December 19 above Diamond Creek. Sampling at National Canyon ceased on December 7 when a helicopter flew into and broke the cableway.

Discharge measurements were made and sediment on the river bed (bed material) was sampled about every other day. Suspended sediment was sampled about twice each day at each gage. Special studies at the National Canyon gage included measurement of bedform movement past the cableway with a recording depthfinder and sampling of sediment moving within 7.6 centimeters of the bed (bedload) with the Helley-Smith sampler (Helley and Smith, 1971). Bed material was sampled with the US BM-54 sampler and suspended sediment with either the US P-61 sampler (Vanoni, 1975, p. 330-332) or the US D-77 sampler (Federal Interagency Subcommittee, 1981). High flow velocities during the sampling period required the use of additional 45.4-kilogram weights on the samplers. Discharge during much of the sampling period was steady and close to the high end of the regulated-flow spectrum (about 730 m^3/s).

The collection of water and sediment data during unsteady flow is planned for a 4-month period beginning October 1, 1985. Because of the short sampling period and daily discharge fluctuations expected, a different sampling scheme is being developed for the period. Parties of six will be stationed at each gage for eight periods of 4 days each. Field parties will begin sampling at Lees Ferry and move downstream as each 4-day period is completed. Discharge measurements will be made and suspended-sediment samples collected over each daily discharge cycle (Fig. 2) during the 4-day period. Rapidly changing stage will require shortening of sampling and measurement time on rising and falling limbs of the daily hydrograph. Because channel configuration and flow velocities in each section are required for computation of discharge of unmeasured sediment load, discharge measurements and suspended-sediment sampling will be combined when stage is changing rapidly. At every location in the cross section where a depth-integrated suspended-sediment sample is collected, total depth and average velocity will be measured. The 1983 bed-material samples have shown that grain-size distribution was relatively stable at each section and fewer samples will be collected in 1985. Because suspended-sediment concentrations are extremely low at Lees Ferry (often less than 10 milligrams per liter), that gage will be sampled for 2 days rather than 4.

Data were collected during the 1983 sampling period at gaging stations on the three largest tributaries in the reach—Paria River at Lees Ferry (09382000), Little Colorado River at Cameron (09401200), and Kanab Creek near Fredonia (09403780) (Fig. 1). At the gaging stations, these tributaries have drainage areas of 3,650, 68,600, and 2,810 km^2, respectively. According to Borland (1962, Sediment inflow into Lake Powell and Lake Mead and effects on the Colorado River channel: unpublished report on file at U.S. Bureau of Reclamation, Engineering and Research Center, Lakewood, Colorado) these three tributaries supplied about 19 percent of the total sediment inflow to Lake Mead before the completion of Glen Canyon Dam. The combined drainage areas are about 52 percent of the total contributing drainage area between Glen Canyon Dam and Hoover Dam.

Suspended-sediment samples were collected twice daily at the gage on the Little Colorado River and periodically during flow events on Kanab Creek and the Paria River. Periodic discharge measurements were made on all three streams to verify existing stage-discharge relations.

All data from the 1983 sampling period have been entered into the USBR computer in Denver and the USGS computer in Tucson. Data in computer files include suspended-sediment concentrations, grain-size distributions of bed material and suspended-sediment samples, and cross-section geometry measured during collection of suspended-sediment samples and measurement of discharge.

Channel-Bed Surveys

Side-scan sonar and seismic-reflection surveys of the channel bed were made in 1984 to collect data on bed material and channel geometry between the gaging stations. This work is covered in greater detail by R. P. Wilson in his paper "Sonar Patterns of Colorado River Bed, Grand Canyon," this volume. A map of bed materials is being prepared from the side-scan sonar data collected on a 10-day raft trip in March 1984, bed-material samples, and depth-finder data. The bed-material samples collected at gaging stations and samples collected between gages by pipe dredge during a raft trip in September 1985 revealed that the side-scan sonar could not reliably distinguish sand from coarser bed material. The side-scan sonar and depthfinder records show that large areas of the bed are deformed into wavelike bedforms with heights from 0.3 to 2 meters. These bedforms were found to be composed primarily of sand and fine gravel. The bed-material map will show areas of bedrock and large boulders and areas of alluvium finer than boulders. The areas of alluvium finer than boulders will be further subdivided into areas of plane bed and areas of wavelike bedforms.

Seismic-reflection and depthsounder surveys of 224 cross sections were made on a 10-day raft trip in April-May 1984. The goal of the surveys was to measure cross-section geometry between gaging stations and to determine the thickness of alluvium, in particular the sand-sized fraction. Multiple reflections from the channel bottom, air entrainment, and lack of penetration of channel-bed sediments, however, combined to give poor results in many cross sections. Estimates of minimum thickness of alluvium will be possible from survey results for some of the measured sections. The estimates will define some limits to the thickness of movable material, which must be determined for the sediment-transport model. Bed-material data and cross-section geometry at the 224 cross sections will be used as input to the sediment-transport model described briefly in the following section and in more detail by C. J. Orvis and T. J. Randle in their paper "Sediment Transport and River Simulation Model," this volume.

DATA ANALYSIS

Analysis of Historical Streamflow Data

Data from the gaging stations at Lees Ferry and near Grand Canyon collected from 1895 to the present will be examined in order to glean information about the long-term dynamics of the Colorado River (D. E. Burkham, Consultant, Carmichael, California, written commun., 1985). Relations of discharge to stage, depth, velocity, and width will be determined, and temporal changes in those relations identified. Changes in the ability of the river to transport water and sediment at a given stage will be identified by the analysis. Temporal variations in scour and fill in the section controlling stage at the gage may affect the ability of the river to transport water and sediment. Examination of the historical data will allow evaluation of trends in scour and fill at the gaged sections. Because historical data include both the preregulation and postregulation periods, the effects of flow regulation on the maximum depth of scour, on trends of scour and fill, and on hydraulic conditions in those sections can be evaluated. The study will document changes in hydraulics of flow in the main channel caused by input of large amounts of sediment by tributaries.

Development of Sand-Load Rating Curves

Suspended-sediment data collected at the gages shown on Figure 1 are being used to develop a relation between water discharge and discharge of sand-sized sediment (sand-load rating curve) for each gaged section. Discharge of total sand load is being computed from sampled suspended-sediment concentrations and channel hydraulics using the modified Einstein method (Colby and Hembree, 1955). Because bed material ranges from bedrock or stable coarse material to predominantly fine sand, each section is divided into subsections according to bed-material composition, and total sand load is computed separately for each subsection. Total sand transport through the section is then obtained by summing results for all subsections. Sand discharges determined from samples of bedload collected at the gages above National Canyon and above Little Colorado River with the Helley-Smith sampler are of the same order of magnitude as the bedload discharge computed with the modified Einstein method. Least-squares linear regression of the logarithms of total sand-load discharge on the logarithms of stream discharge is being used to develop the total sand-load rating curve for each gaging station.

Sand-load rating curves developed from measured suspended-sediment concentrations are being compared to those computed from similar channel geometry and flows with five predictive sediment-transport equations. The Ackers and White equation (Ackers and White, 1973), the Meyer-Peter and Müller relation (Meyer-Peter and Müller, 1948), the Toffaleti equation (Toffaleti, 1969), Einstein method as modified by the Bureau of Reclamation (Einstein, 1950; Pemberton, 1972), and the Yang equation (Yang, 1973) were used. All predictive equations tested to date with the exception of the Meyer-Peter and Müller equation are equally reliable in predicting the sand-load transport. The development of the sediment-rating curves is described in greater detail by E. L. Pemberton and T. J. Randle in their paper "Colorado River Sediment Transport in Grand Canyon," this volume.

An important feature in the application of the sediment-transport model to the Colorado River below Glen Canyon Dam is the influence of tributary inflow to the river. Sediment from tributary streams may be an important source of sand available to the bars. Data from three to five suspended-sediment samples on each of the three sampled tributaries were used, together with section hydraulics, to compute sand load in the unsampled zone using the modified Einstein method. All suspended-sediment samples collected were used to develop a sand-load rating curve for each tributary. New rating curves were compared with historical data at each gage. The computed unsampled sand load was added to the suspended-sand load to determine total load. The total load rating curves were used in the flow-duration, sediment-rating curve method (Miller, 1951) to determine sediment yield from each of the three tributaries. Sediment inflow from ungaged tributaries will be estimated either from yields computed for the three gaged tributaries with a weighting factor or from sediment yields determined in earlier USBR sediment-inflow computations (Borland, W. M., 1962, Sediment load inflow into Lake Powell and Lake Mead and effects on the Colorado River channel).

SEDIMENT-TRANSPORT MODEL

The "Sediment Transport and River Simulation Model" (STARS) described in the report by C. J. Orvis and T. J. Randle, this volume, is a one-dimensional steady-flow model. Unsteady flow can be approximated by changing discharge between time steps. Variations in bed material, geometry, and velocity across the channel can be taken into account by the use of equal-discharge stream tubes aligned parallel to the channel. Water-surface profiles and section hydraulics are computed by the standard step method (Chow, 1964, p. 7-42). Any of several predictive sediment-transport equations can be used to compute sediment discharge at a section from simulated hydraulics at that section. Sediment is routed through the river system in the stream tubes. Input to the model includes initial cross-section geometry and elevation, grain-size distribution of bed-material, thickness of movable material, and hydrograph at the upstream end of the modeled reach. Changes in section geometry and bed-material composition, including bed armoring, are simulated by the model.

The hydraulic portion of the model was tested by duplicating a water-surface profile surveyed in 1923 (Birdseye, 1923). Cross-section geometry and elevation data used are described by R. P. Wilson, this volume. Cross sections concentrated at rapids were added to duplicate the 1923 profile. Cross sections were estimated from top widths measured from 1973 and 1984 photographs and from nearby cross sections measured in 1984 (R. P. Wilson, this volume). Spot checks were also made from surveyed elevations at 21 locations from February to April 1985 by the National Mapping Division of the Geological Survey. Bed-material samples collected at the five gaging stations, along with bed samples described in

Wilson's report, this volume, were used to identify bottom material in the cross sections. The channel bottom upstream from and through each rapid probably is armored with material of gravel-to-cobble size or bedrock.

The model will be calibrated first for the reach from the gage at Lees Ferry to the mouth of the Little Colorado River, a distance of about 98 km, using data from June 1983 to December 1983. When satisfactory calibration is obtained for that reach, the model will be calibrated for the remaining three downstream reaches. The model will be verified with data from the 1985 unsteady-flow sampling period and recalibrated if necessary. The calibrated sediment model will be used with a range of input hydrographs representative of feasible Glen Canyon Dam flow releases. Transport of sand-sized sediment through the study reach and net gain or loss of sand in each of the four reaches between gaging stations will be computed for each alternative.

BAR-STABILITY STUDIES

The nature and cause of sand-bar and bank changes along the Colorado River between the gages at Lees Ferry and above Diamond Creek will be investigated in two phases. In the first phase, long-term changes in bars will be documented and systematized by examination of available photographs and surveys. Changes will be related to values of geomorphic variables measured from photographs or easily measured in the field. An attempt also will be made to relate time trends in bar changes to known changes in water and sediment discharge in the Grand Canyon over similar periods.

Available data will be examined in order to quantify, to the degree possible, the change in location and size of sand bars and other components of the river banks prior to and subsequent to completion of Glen Canyon Dam. Certain aspects of this issue already have been investigated (Brian and Thomas, 1984; Turner and Karpiscak, 1980; Beus and others, 1984). An attempt has not been made to integrate all available data, to identify patterns of change in size or location of bars along the channel, or to relate changes in bars to (1) local geomorphic characteristics that may control hydraulic conditions during different flows; (2) intermediate scale characteristics of various long reaches of the river, such as distance from the dam; and (3) time-dependent data, such as flow or sediment transport. The effort of the first phase is intended to (1) identify the geomorphic characteristics that influence bar location, size, and shape; (2) determine the propensity for bars and banks of different types to change over time; (3) determine the degree to which bar and bank changes can be related to location, geomorphic characteristics, changes in main-stem flow and sediment transport, or random phenomena such as tributary flash floods. An important interpretation of the data will be the determination as to which changes are systematic and which changes are not characteristic of system-wide response. A classification of existing bars will also be developed using interpretation of the 1984 photographs supplemented by fieldwork. The classification will be based on such characteristics as bar geometry, geometry of flow constrictions and the features that cause them, and location of bars relative to eddies and flow constrictions.

In the second phase, processes currently acting on bars will be identified and quantified so that short-term bar changes can be related to different observed regulation regimes, and general aspects of sand erosion, transport and deposition can be understood. Results of this phase will distinguish and quantify bar changes not attributable to main stem river processes. Exploratory data collection was begun in the summer of 1984, and the most of the data will be collected between April 1985 and January 1986.

In this phase, physical processes active in shaping sand bars will be identified and measured in selected short reaches of the Grand Canyon. The magnitude and frequency of various processes will be estimated using direct and indirect measurements and interpretation of geologic data observed in the field. The goal of this phase is to understand how fluctuations in river-related processes affect the intensity and location of sand-bar erosion and deposition. Studies in each short reach will include the following: Geomorphic and surficial mapping of bar and bank material; repeated surveying and mapping of prominent bars and nearby areas; description of sedimentary characteristics of bars and interpretation of apparent flow conditions of deposition and reworking; observation and measurement of wave phenomena in eddies; and measurement of flow-velocity distribution in selected eddies and in the main channel.

SUMMARY

Activities of the sediment subteam of the Glen Canyon Environmental Study include measurement and analysis of sediment transport in Grand Canyon National Park, characterization of bed configuration and bed materials along the channel, development of a geomorphic classification of alluvial sand bars, identification of processes that act to modify bars, and development of a predictive sediment-transport model. Study results will be evaluated to assess the effects of the present operating schedule and alternative schedules on sediment transport and storage along the Colorado River in the park. Knowledge of the expected effects then can be considered when management decisions of dam operation are made.

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CHANNEL EVOLUTION IN MODIFIED TENNESSEE CHANNELS

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ABSTRACT

Modification of alluvial channels in West Tennessee creates heightened energy conditions along main stems and tributaries and initiates longitudinal trends of channel adjustment. Changes in bed-level are functions of the magnitude and extent of the imposed disturbance and the location of the adjusting reach in the fluvial network.

Streambed degradation at a site is described by simple power equations. Computed exponents define the magnitude of downcutting with time and decrease nonlinearly with distance upstream from the area of maximum disturbance (AMD). Aggradation begins immediately in reaches downstream of the AMD and in upstream reaches following over-adjustment by the degradation process. Aggradation rates increase linearly with distance downstream from the AMD and can be estimated from local degradation rates.

Channel widening by mass wasting follows degradation and continues through aggradational phases. Piping in the loess-derived bank materials enhances bank-failure rates by internally destabilizing the bank. Development of the bank profile is defined in terms of three dynamic and observable surfaces: (1) vertical face (70° to 90°), (2) upper bank (25° to 50°), and (3) slough line (20° to 25°). The slough line develops through additional flattening by low-angle slides and fluvial reworking, and is the initial site of re-establishing riparian vegetation and stable bank conditions.

A six-step, semi-quantitative model of channel evolution in disturbed channels was developed by quantifying bed-level trends, and recognizing qualitative stages of bank-slope development.

INTRODUCTION AND PURPOSE

Alluvial channels adjust to imposed changes in order to offset the effects of those changes and approach quasi-equilibrium. Lane (1955) describes this general balance in terms of the stream power equation:

$$QS \propto Q_s d_{50} \qquad (1)$$

where Q = water discharge, in cubic meters per second;
S = channel gradient, in meters per meter;
Q_s = bed-material discharge, in kilograms per cubic meter; and
d_{50} = median grain size of bed-material load, in millimeters.

Dredging and straightening (shortening), alters both channel cross-sectional area and channel gradient (S) such that they are increased. By equation 1, this results in a proportionate increase in either bed-load discharge (Q_s), or bed-material size (d_{50}), or both, such that rapid and observable morphologic changes occur.

Channelization (dredging and straightening) is a a common engineering practice, aimed at controlling flooding or draining wetlands. Quantification of subsequent channel responses can be valuable in estimating their effects on river-crossing structures and lands adjacent to these channels. The purposes of this study are to (1) assess the channel changes and network trends of bed-level response following modifications between 1959 and 1972 to alluvial channels in West Tennessee (fig. 1), and (2) to develop a conceptual model of bank-slope development to qualitatively assess bank stability and potential channel widening.

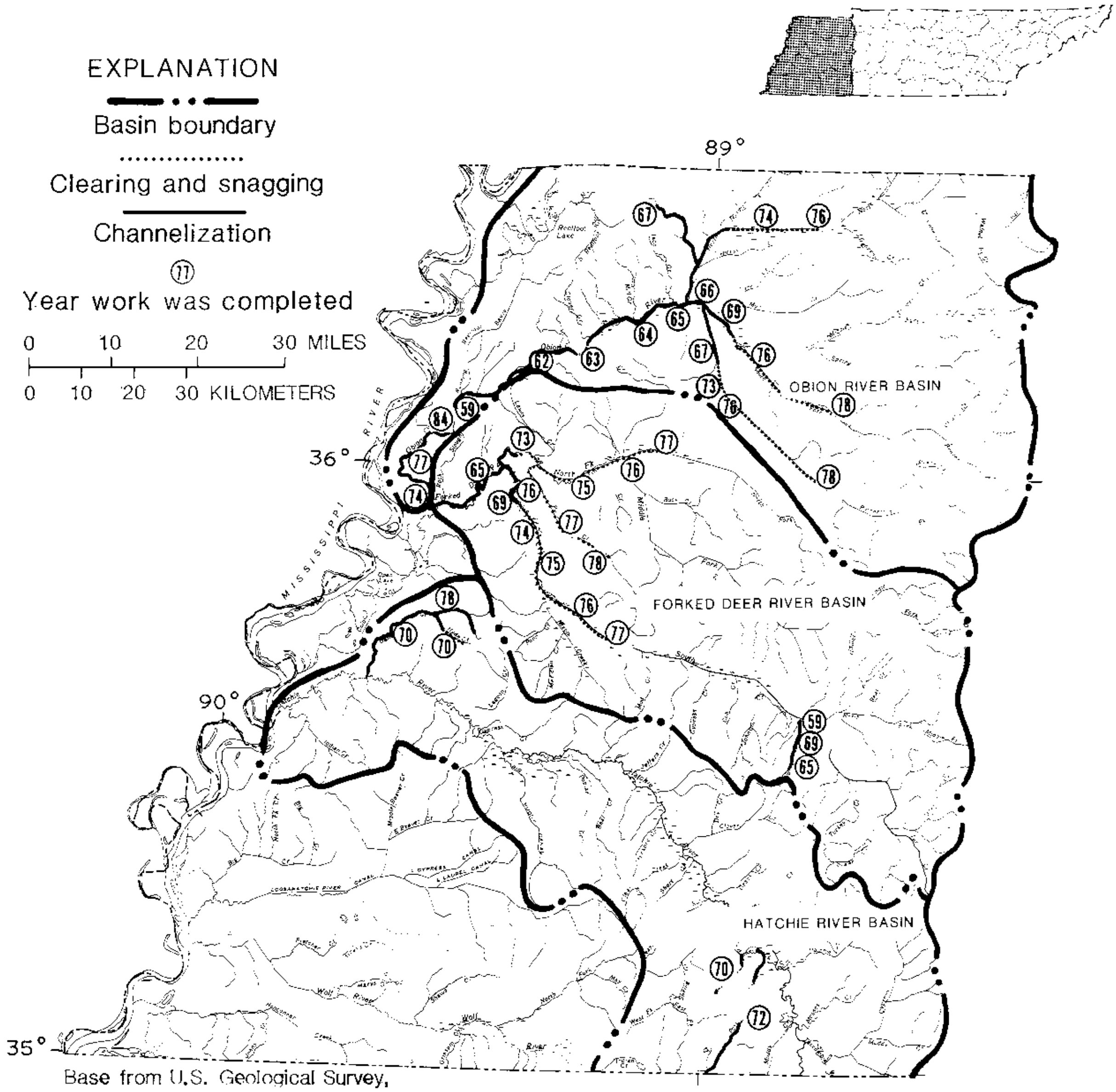

Fig. 1. Extent of channel modifications in West Tennessee.

STUDY AREA

The West Tennessee region is an area of approximately 27,500 square kilometers bounded by the Mississippi River on the west and the Tennessee River on the east (fig. 1). All of the stream systems studied drain to the Mississippi River via the Obion, Forked Deer, and Hatchie River basins (fig. 1). These rivers are cut into unconsolidated and highly erosive formations (U.S. Department of Agriculture, 1980), predominantly of Quaternary age. Wisconsin loess dominates the surficial geology of the region and the majority of the channels have medium-sand beds. These alluvial channels are free to systematically adjust their profiles following a disturbance due to a lack of bedrock control of local base level.

DATA COLLECTION

Channel morphology data were collected and compiled from previous surveys to determine channel change with time. These data consisted of bed elevations and gradients, channel top-widths, and channel lengths before, during, and after modification. Where available, gaging station records were used to record annual changes in the water-surface elevation at a given discharge (specific gage, Robbins and Simon, 1983). Changes in the water-surface elevation at the given discharge imply similar changes on the channel bed and can be used to document bed-level trends (Blench, 1973; Robbins and Simon, 1983).

The identification and dating of various geomorphic surfaces can be diagnostic in determining the relative stability of a reach and the status of bank-slope development (Simon and Hupp, 1985; Hupp and Simon, this volume). Data collection involved the locating and dating of riparian vegetation on (1) newly stabilized surfaces to determine the timing of initial stability for that surface, and on (2) unstable bank surfaces to estimate rates of bank retreat (Hupp and Simon, this volume).

BED-LEVEL RESPONSE

Adjustments in modified West Tennessee channels and their tributaries include incision, headward erosion, downstream aggradation, and bank instabilities. Previous investigations (Robbins and Simon, 1983) identified adjustments to gentler gradients and reduced energy conditions. These studies provided empirical time-based relations of site-specific gradient adjustment. These studies further determined that the bed of the Hatchie River had remained stable during the period when streams of the Obion and Forked Deer River basins were undergoing drastic morphologic changes.

Bed-level adjustment trends at gaged sites are determined by calculating the mean annual specific-gage elevation for the period just prior to, and following channelization activities (Blench, 1973; Robbins and Simon, 1983). Specific-gage trends serve as examples for the ungaged, but periodically surveyed sites (Simon, in process). The processes of aggradation and degradation at a site, through time, are described by simple power equations that take the general form:

$$E = a\,(t)^b \qquad (2)$$

where E = elevation of the bed or specific gage for a given year, in meters above sea level;
a = pre-modified elevation of the bed or specific gage, in meters above sea level;
t = time since channel work, in years, where $t_0 = 1.0$; and
b = exponent determined by regression, indicative of the nonlinear rate of change on the bed.

Equation 2 reflects bed-level response at a site (table 1) and implies that adjustment rates are initially rapid and then diminish as the bed elevation asymptotically approaches condition of no net change (fig.2). The magnitude of b denotes the nonlinear rate of degradation, (negative b), or aggradation (positive b) and is used as the dependent variable in regression analyses for determination of longitudinal adjustment trends.

Table 1. Sites with calculated gradation rates (b).

[Note: b, non-linear gradation rate; n, number of observations; r2, coefficient of determination; RKM, river kilometer; T0, start of observed gradation process; *, specific gage data used]

Stream	b	n	r2	RKM	T0
Cane Creek					
	-0.01620	7	0.99	23.89	1969
	0.00168	2	1.00	23.89	1980
	-0.02022	4	1.00	20.24	1969
	0.01052	2	1.00	20.24	1980
	-0.03300	3	1.00	14.46	1969
	0.00770	2	1.00	14.46	1980
	-0.03131	2	1.00	10.09	1969
	0.00352	2	1.00	10.09	1980
	-0.04126	3	0.91	6.53	1969
	-0.02011	4	0.92	4.06	1969
	0.00835	2	1.00	4.06	1980
Cub Creek					
	-0.00243	3	0.69	11.13	1969
	-0.00342	3	0.87	9.22	1969
	-0.00565	4	0.88	3.48	1969
	-0.00905	5	0.91	2.48	1969
	0.00272	2	1.00	2.48	1976
Hoosier Creek					
	-0.00843	3	1.00	8.29	1967
	-0.01130	4	0.94	4.81	1966
	-0.02081	3	0.67	0.88	1965
	0.00274	2	1.00	0.88	1968
	-0.02630	3	0.99	0.02	1965
Hyde Creek					
	0.00281	2	1.00	3.81	1975
	-0.00737	2	1.00	3.81	1969
	-0.01070	4	0.92	2.22	1969
	-0.01380	3	0.99	1.19	1969
	-0.02050	4	1.00	0.02	1969
Meridian Creek					
	-0.00326	3	0.99	5.94	1965
	-0.00580	4	0.98	4.73	1964
	-0.00341	3	0.99	2.41	1969
	-0.00190	3	0.99	1.54	1967
North Fork Forked Deer River					
	-0.00740	4	0.95	38.46	1977
	-0.01076	5	0.52	32.47	1974
	-0.00839	4	0.96	30.28	1978
	-0.01720	10	0.95	8.53	1973*
	-0.02297	3	0.87	6.16	1972
North Fork Obion River					
	0.00111	15	0.69	59.37	1969*
	-0.00206	2	1.00	42.48	1979
	-0.00490	2	1.00	33.95	1975
	-0.00372	13	0.80	28.96	1972*
	-0.01240	6	0.93	15.83	1965*
	-0.02470	4	0.85	9.49	1965*
	0.00303	5	0.89	9.49	1967*
Obion River					
	-0.02220	10	0.95	110.22	1965*
	0.00463	10	0.74	110.22	1974*
	-0.04030	4	0.81	100.08	1965*
	0.00235	16	0.76	100.08	1968*
	0.00908	19	0.93	86.40	1965*
	0.00518	15	0.84	55.03	1963*
	0.00585	16	0.74	33.47	1960*
Pond Creek					
	-0.00828	5	0.81	18.29	1977
	-0.00799	4	0.84	15.80	1977
	-0.01233	4	0.97	11.78	1977
	-0.00900	5	0.79	1.71	1977
Porters Creek					
	-0.01069	7	1.00	27.51	1971
	-0.01320	7	0.99	18.02	1971
	-0.00578	6	1.00	14.30	1971
Rutherford Fork Obion River					
	0.00149	19	0.60	48.11	1965*
	-0.00317	4	0.91	28.80	1977
	-0.00493	3	1.00	24.46	1977
	-0.00991	4	0.79	16.73	1972
	0.00356	4	0.99	16.73	1977
	-0.01728	9	0.93	7.88	1965*
	0.00433	9	0.88	7.88	1974*
Soutn Fork Forked Deer River					
	-0.00895	6	0.59	44.41	1976
	-0.00950	10	0.92	26.23	1974*
	-0.00978	5	0.76	21.40	1969
	-0.01264	5	0.96	19.15	1969
	-0.01630	15	0.94	12.71	1969*
	0.01180	13	0.92	5.31	1969*
South Fork Obion River					
	-0.02430	11	0.87	9.33	1965*
	0.00544	9	0.88	9.33	1975*
	0.00133	13	0.90	55.35	1969*
	-0.00054	4	0.26	45.70	1972
	-0.00238	6	0.50	37.33	1972
	-0.00661	7	0.90	30.89	1977*
	-0.00573	5	0.87	27.03	1972
	-0.00932	4	0.94	18.34	1972

Degradation exponents (-b) generally range from -0.005 to -0.040, with the greatest rates of change occurring near the upstream side of the area of maximum disturbance (AMD), usually the upstream terminus of the channel work. Increases in downstream channel gradient and cross-sectional area by man, results in a stream power that is more than sufficient to transport the bed-material delivered from upstream. The bed of the channel therefore erodes headward to increase bed-material transport and (or) reduce channel gradient. Typically, degradation occurs for 10 to 15 years at a site and can deepen the channel up to 6.1 meters (fig. 2).

Aggradation rates (+b) are less than their degradation counterparts, and generally range from 0.001 to 0.009 with the greatest rates occurring near the stream mouth. This process begins immediately following channelization downstream of the AMD, may reach 0.12 meters per year, and can continue for more than 20 years. As degradation proceeds upstream from the AMD to reduce channel gradient, aggradation downstream of the AMD similarly flattens gradients as part of an integrated basin response due to the man-induced increases in energy conditions (Simon, in process).

Aggradation also occurs at sites upstream of the AMD that have been previously degraded and suggests an initial overadjustment by degradation. This secondary aggradation occurs as headward infilling. The rate of infilling at a site is approximately 78 percent less (S_E = 2.85) than the initial rate of degradation at that site (Simon, in process). Expected aggradation rates at sites upstream of the AMD can therefore be estimated at approximately 22 percent of the degradation rate at the site in question. Equations have been developed to calculate aggradation rates in the Cane Creek, Forked Deer, and Obion River systems and are reported in Simon (in process).

Model of Bed-Level Adjustment

Degradation rates decrease nonlinearly with distance upstream from the AMD in the Cane Creek, Forked Deer, and Obion River basins as values of b become less negative and approach 0.0 (fig. 3). Linear relations are derived from these trends (fig. 3) and used to predict degradation rates in the three basins (Simon, in process). Similar relations were derived for the 13 individual streams studied and are reported in Simon (in process).

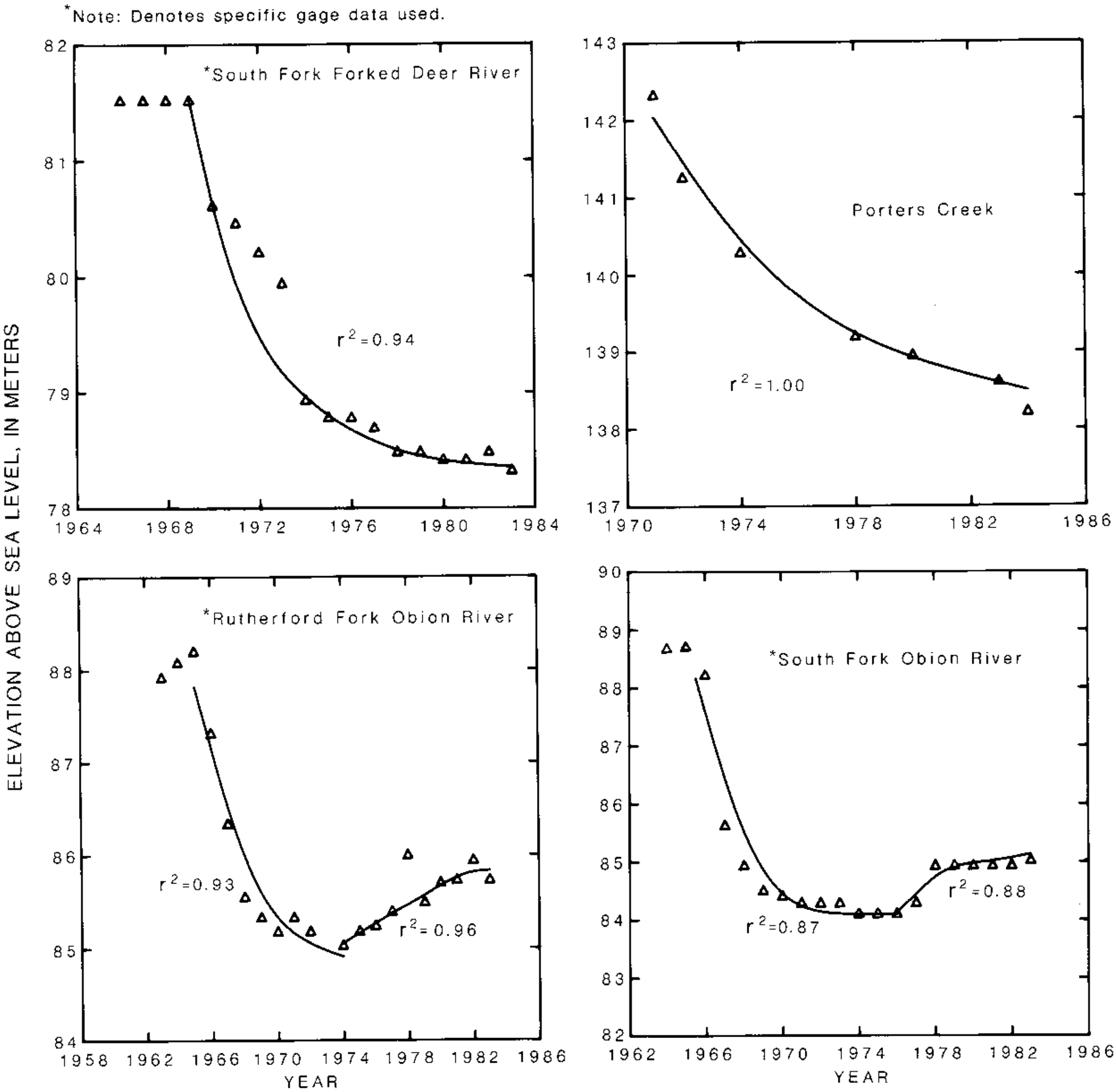

Fig. 2. Examples of fitting power equations to degradation and aggradation trends through time.

Completed and ongoing rates of bed-level change as a function of distance upstream from the mouth of the Obion River are shown in figure 4. Rates of bed-level change are plotted on the y-axis; the horizontal 0.00 line represents the threshold of critical power and, theoretically, no net change on the channel bed (Bull, 1979). Deviations from the 0.00 line denote a period of bed-level change caused by the imposed disturbance or by subsequent channel responses. The Obion River is used as a typical example of bed-level response. Similar models have been documented for other streams and basins in West Tennessee (Simon, in process).

The modifications imposed on the Obion River can be considered to have caused a rejuvenation of the fluvial network. A condition of peak disequilibrium occurred at the upstream end of the channel work at its completion in 1967. Point 'A' in figure 4 denotes the location of this peak disturbance (AMD), represents maximum rates of degradation, and serves as a starting time (1967) for a space for time substitution. Incision upstream according

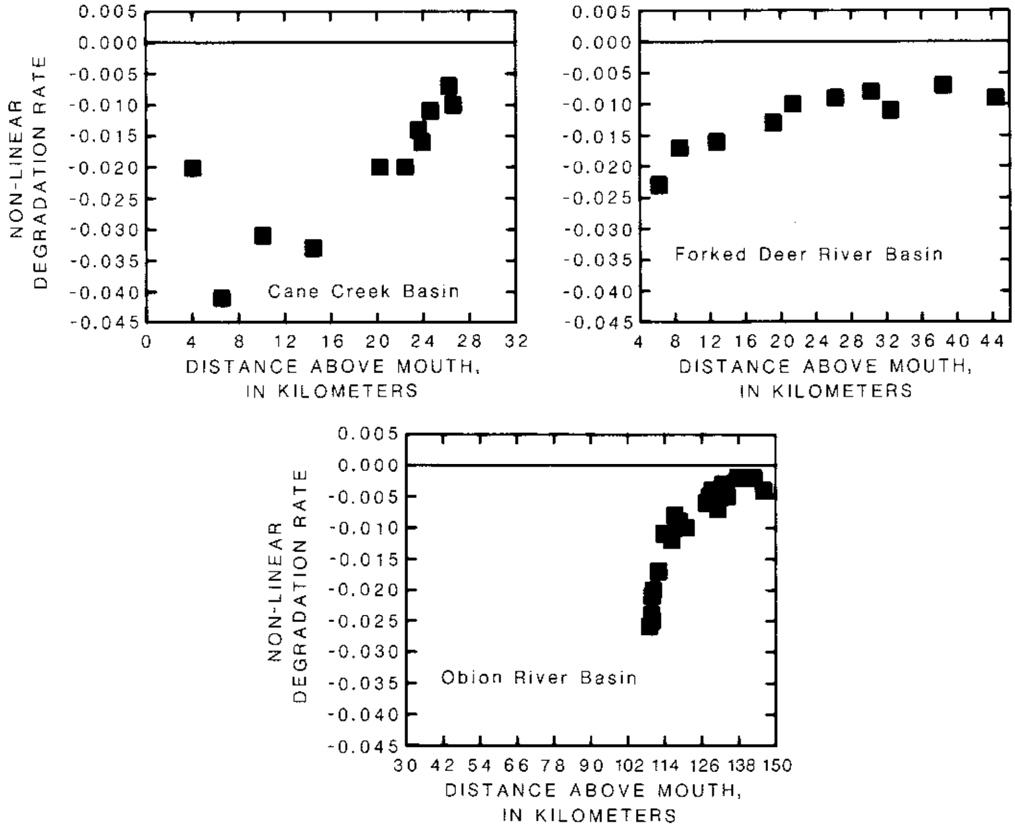

Fig. 3. Longitudinal trends of degradation in three West Tennessee networks.

to line 'C' and aggradation downstream, according to line 'B', progress from this point in time and space (fig. 4).

Degradation for 10 to 15 years at sites just upstream of 'A' (fig. 4) reduces gradients to such an extent that the river can no longer transport the loads delivered from degrading reaches further upstream. These sites then experience secondary aggradation due to the overadjustment by degradation processes ('D' in fig. 4). This headward infilling occurs at magnitudes 78 percent less than the previous degradation rate and suggests an overcompensation of 22 percent and oscillatory channel response (Alexander, 1981). Expected shifts to secondary aggradation at sites further upstream that are presently degrading are provided by the inclusion of estimated data and serve to extend line 'B-D' in time and space (fig. 4).

Where lines 'C' and 'D' approach the 0.00 line (fig. 4), gradation processes due to downstream modifications are minimized, and sites will not degrade. This is estimated to take place near river kilometer 145 in the Obion River system. Upstream of the estimated intersection of the 0.00 line at 'E', aggradation continues to take place at low, premodified rates, according to 'natural' basin and channel characteristics. Channel conditions far upstream of the AMD near 'E' represent not only moderate adjustment processes and premodified aggradation rates at those sites, but also conditions that will be attained by downstream reaches in the future as the channel network approaches quasi-equilibrium.

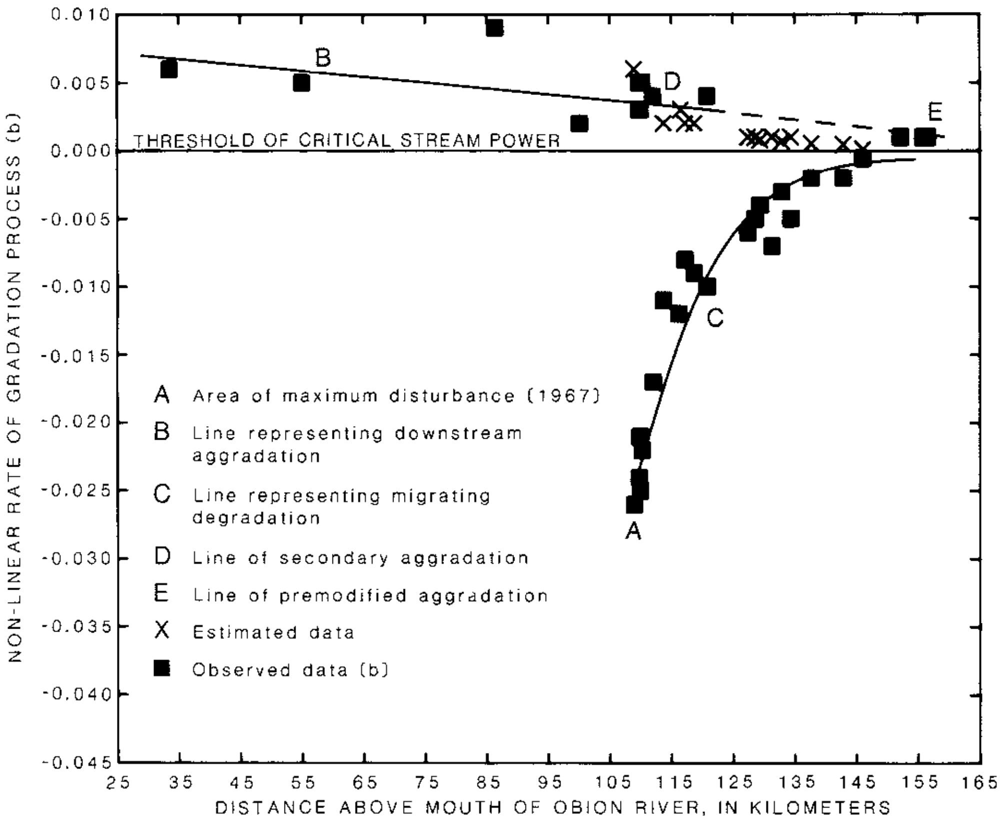

Fig. 4. Model of bed-level response for the Obion River.

WIDTH ADJUSTMENT AND BANK-SLOPE DEVELOPMENT

Channel widening (DW) and bank failure by mass-wasting processes are common attributes of adjusting channels in West Tennessee. Bank failure is induced by the overheightening and oversteepening of the bank by degradation and by undercutting at the toe of the bank (Thorne and others, 1981). Piping in the loess-derived bank-materials enhances bank failure by internally destabilizing the bank.

The effect of bed-level lowering by degradation on channel widening is reflected by the following direct correlation between the degradation expoint (-b) and DW where $r^2 = 0.50$, $S_L = 0.0001$, and $n = 49$. This relation implies that the lower degradation rates headward reduce the tendency for channel widening upstream (Simon, in process). Changes in channel width (DW) following the most recent major channel modifications imposed on West Tennessee streams are summarized in table 2. Mean DW values for each stream are merely a

Table 2. Range of changes in channel top width (DW) by stream.

	DW, in meters					Years since most recent major channelization (from 1983)
	Mean	n	S	Min	Max	
Cane Creek	18.2	6	6.2	14	25	13
Cub Creek	2.1	4	5.1	0	6.7	13
Hoosier Creek	14.2	3	21.2	7.0	27	16
Hyde Creek	3.1	3	5.0	.3	5.4	18
Meridian Creek	3.4	2	3.0	2.4	4.3	16
North Fork Forked Deer River	10.3	4	16.9	3.0	25	10
North Fork Obion River	10.4	5	15.4	.6	25	16
Obion River	37.8	4	26.8	20	59	17
Pond Creek	2.9	3	1.7	2.1	4.3	5
Porters Creek	7.7	3	7.5	3.7	12	11
Rutherford Fork Obion River	7.2	5	11.4	0	18	14
South Fork Forked Deer River	13.6	6	11.5	4.9	27	14
South Fork Obion River	11.9	6	14.5	0	36	14

point of reference and not a precise average of width adjustment, due to the variability of gradation processes and the relative location of sites along a given stream. Minimum and maximum values of DW (table 2), however, do reflect a realistic range of width changes throughout the stream lengths studied.

Processes and Stages of Bank Retreat and Slope Development

The processes and successive forms of bank retreat and bank-slope development reflect the interaction of hillslope and fluvial processes. Interpretations of these processes and forms are based largely on bed-level adjustment trends and botanical evidence (Simon, in process). An idealized representation of six stages of bank retreat and bank-slope development is provided in figure 5. The stages represent distinguishable bank morphologies characteristic of the various site-types that describe bed-level adjustment (table 3).

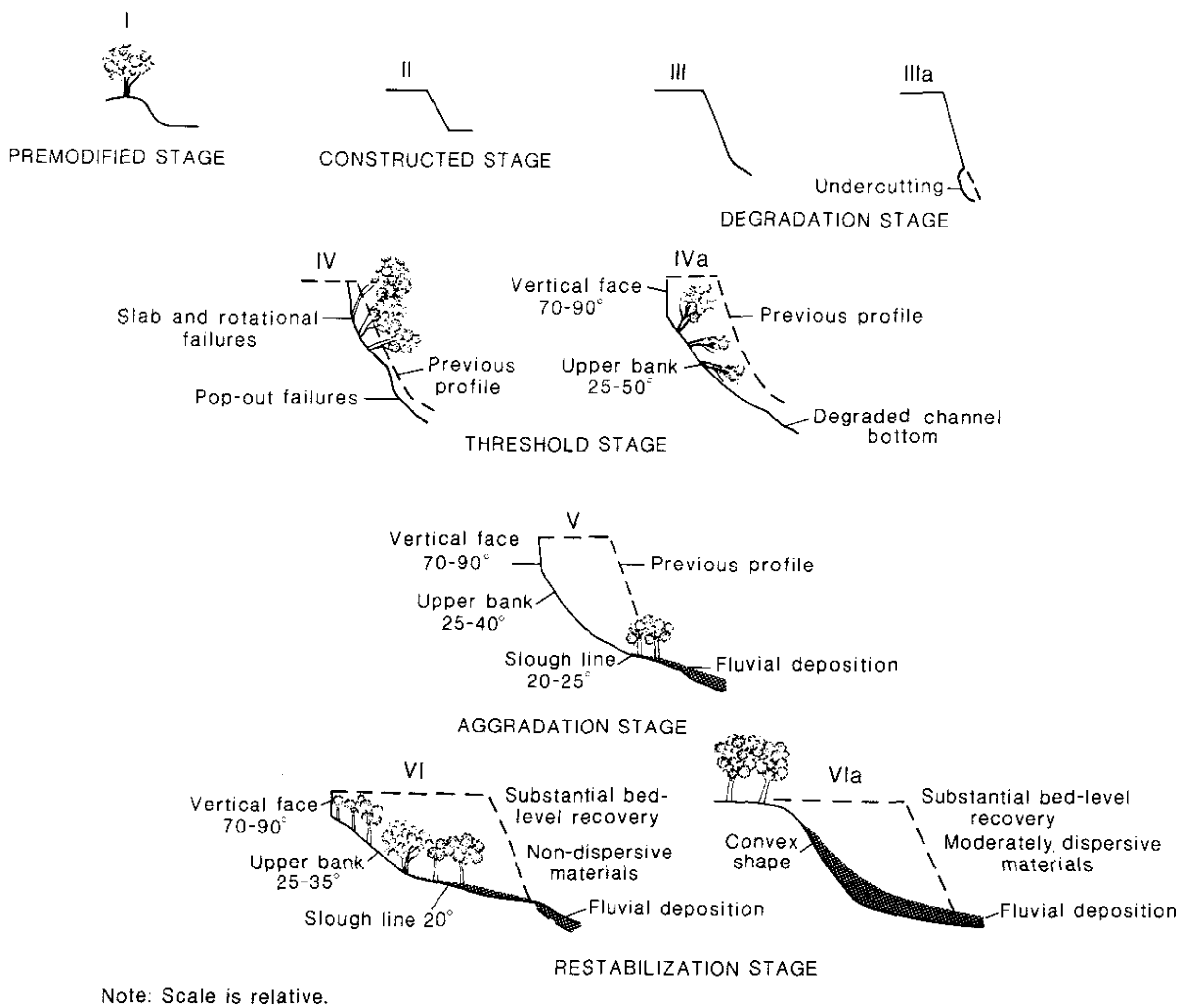

Fig. 5. The six stages of bank-slope development.

Table 3. Stages of bank slope development.

Stage No.	Stage Name	Bed-level adjustment type	Location in network	Process on channel bed	Active widening	Failure types	Bank surfaces present	Approximate bank angles, in degrees
I	Premodified	Premodified	Upstream-most reaches.	Transport of sediment or mild aggradation.	No	---	---	20-30°
II	Constructed	---	Where applicable	Dredging	By man	---	---	18-34°
III	Degradation	Migrating degradation.	Upstream from the AMD[1].	Degradation	No	---	---	20-30°
IV	Threshold	Migrating degradation.	Close to the AMD[1].	Degradation	Yes	Slab rotational pop-out.	Vertical face upper bank.	70-90° 25-50°
V	Aggradation	Secondary aggradation.	Upstream of the AMD[1].	Aggradation	Yes	Slab rotational pop-out low angle slides.	Vertical face upper bank slough-line.	70-90° 25-40° 20-25°
VI	Restabilization	Downstream-imposed aggradation.	Downstream of the AMD[1].	Aggradation	No	Low-angle slides pop-out.	Vertical face upper bank slough-line.	70-90° 25-35° 15-20°

[1]AMD = area of maximum disturbance.

Premodified Stage (I)

Premodified bank conditions are assumed to be the result of "natural" land-use practices and fluvial processes. Bank failure by mass-wasting processes generally does not occur and banks are considered stable. Premodified bank conditions (stage I in fig. 5) are characterized generally by low-angle slopes (20° to 30°), convex upper bank and concave lower bank shapes, and established woody vegetation along the top bank and downslope towards the low-flow channel (fig. 5). Stage I channel widths may narrow slowly with time due to mild aggradation and bank accretion. Sand often is found deposited on bank surfaces. Limited channel widening through bank caving and fluvial erosion may occur on some outside meander bends.

Constructed Stage (II)

The construction of a new channel involves reshaping of the existing channel banks or the repositioning of the entire channel. In either case, the banks are generally steepened, heightened, and made linear (stage II in fig. 5). Modified West Tennessee channels generally are constructed as trapezoids with bank slopes ranging from 18° to 34°. Channel widths are increased and vegetation is removed in order to convey greater discharges within the channel banks.

Degradation Stage (III)

The degradation stage (stage III in fig. 5) is characterized by the lowering of the channel bed and the consequent increase in bank heights. Downcutting generally does not steepen bank slopes directly but maintains bank angles close to the angle of internal friction (Skempton, 1953; Carson and Kirkby, 1972; and Simon and Hupp, 1985). Steepening occurs when moderate flows attack basal surfaces and remove toe material along the heightened banks (stage IIIa in fig. 5). Ideally, widening by mass wasting does not occur because the critical bank height has not yet been exceeded. The degradation stage ends when the critical height of the material is reached.

Threshold Stage (IV)

Stage IV (fig. 5) is the result of continued degradation and basal erosion that further heightens and steepens the channel banks. The critical bank height is exceeded, and bank slopes and shapes become the product of mass-wasting processes. Slab failures occur due to excessive undercutting and the loss of support for the upper part of the bank. Pop-out failures at the base of the bank due to saturation and pore-water pressure (Thorne and others, 1981) similarly oversteepen the upper part of the bank. Deep-seated rotational failures shear along a circular arc and often become detached from the top bank surface by piping and tension cracking. These failures can leave 1.8- to 3-meter long slickensides along the failure plane and have been observed up to 60 meters long and 6 meters wide (Simon and Hupp, 1985). The bank also can fail as plates along a series of pipes that form in the bank mass and intersect the flood plain surface (Simon, in process). Occasional debris avalanching occurs as well. These failure mechanisms are the most intense in highly dispersive loess materials, such as in the Cane Creek basin.

The failed material that comes to rest on the bank forms a definable surface at slopes ranging from 25° to 50° (stage IV in fig. 6). This surface is termed the "upper-bank" (Simon and Hupp, 1985) and can often be identified on field inspection by tilted and fallen vegetation. The 'vertical face', representing the top section of the major failure plane, is developed concurrently (stage IV in fig. 6). The vertical face actually may range from 70° to 90°, and represents the primary location of top-bank retreat.

The threshold stage is the first of two that are dominated by active channel widening. Failed material may be carried off by moderate to high flows, thereby retaining the overheightened and oversteepened bank profile, and giving the banks a scalloped appearance.

Aggradation Stage (V)

Stage V (fig. 5) is marked by the onset of aggradation on the channel bed and often can be identified by deposited sand on bank surfaces. Bank retreat dominates the vertical face and upper bank sections, because bank heights still exceed the critical height of the material. The failed material on the upper bank is subject to low-angle slides resulting from continued wetting of the material by rises in stage. This process flattens and extends the upper bank downslope. Older masses of failed material on the upper bank also move downslope by low-angle slides. Such masses show evidence of fluvial reworking and deposition low on the upper bank and create a low-angle surface (20° to 25°) termed the 'slough line', extending downslope from the upper bank (Simon and Hupp, 1985; Hupp and Simon, this volume). Woody vegetation re-establishing on the slough line have been used to date the timing of renewed bank stability along several streams (Hupp and Simon, this volume) (stage V in fig. 5).

A slow and prolonged period of bed-level recovery (as with the loess bed channels) maintains bank heights greater than the critical height of the material for extended periods. Dispersion and tension cracking continue to weaken the vertical face. Parallel bank retreat along the vertical face, and flattening of the upper bank and slough line may continue as the channel creates a new flood plain at an elevation lower than the previous one. The previous flood plain would then become a fluvial remnant, or terrace, over the geologic span of time. This scenario of flood plain development is probably appropriate only for the most highly disturbed channels that are cut through deposits of dispersive loess, such as Cane Creek (Simon and Hupp, 1985). Stage V would then represent the final stage of bank-slope development in these types of channels.

Restabilization Stage (VI)

The restabilization stage is marked by the significant reduction of bank heights by aggradation on the channel bed and by fluvial deposition on the upper bank and slough-line surfaces. Bank retreat along the vertical face by intense mass-wasting processes subsides, due to the lack of bank height relative to the critical height.

Cohesive banks along Porters and Cub Creeks, and areas of the Obion River system that are composed of strongly nondispersive materials, can maintain a vertical face even though the surface is frequently wet by rises in stage. Woody vegetation extends to the base of the vertical face in these cases, and the old flood plain surface becomes a terrace (stage VI, fig. 5). In channels where bank materials are only moderately dispersive and where bed-level has sufficiently recovered to cause greater flow frequencies on the vertical face, the uppermost section of the bank may take a convex shape due to fluvial reworking and deposition (stage VI, fig. 5). In some extensively aggrading downstream reaches, such as along Obion River main stem, the flood plain surface maintains its role as a conduit for moderately high flows, and woody vegetation becomes re-established at the top of the bank and on the flood plain surface (Hupp and Simon, this volume).

The six-stages of bank-slope development represent a conceptual model of width adjustment. Stages are induced by a succession of interactions between gradation and hillslope processes (table 3). The model does not imply that each adjusting reach will undergo all six stages over the course of the adjustment, but that specific trends of bed-level response result in a series of mass-wasting processes, and definable bank forms. The conceptual framework of the simultaneous retreat of the vertical face, and flattening along surfaces below it, is supported by the observations of other investigations reported in Carson and Kirkby (1972, p. 184).

CONCLUSIONS

Channel dredging and straightening between 1959 and 1972 in West Tennessee caused a series of morphologic changes along modified reaches and tributaries. Degradation occurs for 10 to 15 years at sites upstream of the area of maximum disturbance (AMD) and can lower bed-level by as much as 6.1 meters. Aggradation downstream of the AMD can reach 0.12 meters per year with the greatest rates near the mouth. Initially degraded sites experience a secondary aggradation phase in response to excessive incisement and gradient reduction.

The adjustment of channel width is characterized by six stages of bank-slope development: premodified, constructed, degradation, threshold, aggradation, and restabilization. Downcutting and toe removal during the degradation stage causes bank failure by mass-wasting processes when the critical height of the bank material is reached (threshold stage). Top-bank widening continues through the aggradation stage as the slough line develops as an initial site of lower bank stability. The development of the bank profile is defined in terms of three dynamic surfaces: (1) vertical face (70° to 90°), (2) upper bank (25° to 50°), and (3) slough line (20° to 25°).

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VEGETATION AND BANK-SLOPE DEVELOPMENT

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ABSTRACT

Most streams in West Tennessee have been straightened and dredged for flood control. These modifications have initiated long-term changes in bed level and, in many cases, channel widening that occurs predominantly by mass wasting of banks. Dendroecologic and dendrogeomorphic analyses of riparian vegetation yield information on dates and rates of bank failure, bank-sediment accretion or erosion, and degree of bank-slope development. Widening rates up to 2.4 meters per year have been documented along highly unstable reaches. Similar analyses of buried tree stems on re-stabilizing banks indicate average accretion rates of 5 to 7 centimeters per year. Presence of particular species indicate trends of increasing or decreasing stability. The most common woody plants on adjusting bank slopes include black willow, boxelder, river birch, and sycamore. The establishment of riparian vegetation promotes bank stability by ameliorating bank-erosional processes. Banks largely devoid of establishing woody vegetation are characteristic of the most unstable reaches.

INTRODUCTION

Riparian vegetation provides immediate information regarding bank-slope conditions unavailable through standard hydrologic or geomorphic methods. The term riparian, in this paper, refers to vegetation growing on or near stream banks but does not include flood-plain vegetation. Woody plants rapidly establish on most riparian surfaces, except where bank widening rates exceed a minimal time necessary for germination and growth. Trees and shrubs on unstable riparian surfaces exhibit the effects of channel widening through stem and wood deformations formed in response to bank failure. Woody plants on accreting bank surfaces become partially buried, indicating rates of sediment deposition above the initial level of germination or congested adventitious root zones. Species presence and stem density allow inference of relative degrees of stability on different bank features and along different stream reaches.

The purposes of the present study are (1) the presentation of initial results of an interdisciplinary research effort to estimate and interpret the effects of channel modifications, as determined through analysis of riparian woody vegetation on selected West Tennessee streams, and (2) demonstration of the utility of dendrogeomorphic and plant ecologic analyses for the estimation of bank stability, channel widening rates, and rates of bank-sediment deposition and erosion.

Radial tree growth forms a cylinder of new wood each year and, when viewed in cross-section, is called a tree ring. A count of tree rings from increment cores or cross-sections yields the age of the stem at the sample point, so long as each year is represented by a single ring. Trees and shrubs affected by either mass wasting or bank accretion can be cored (with an increment borer) or cross-sectioned to determine dates of bank failure, vegetation establishment, or sediment deposition, through tree-ring counts. Sigafoos (1964), Phipps (1967), Yanosky (1982), and Hupp (1983) have described tree growth in relation to hydrogeomorphic study. Analysis of woody vegetation affected by bank-sediment deposition or erosion has allowed the estimation of rates of these processes (Sigafoos, 1964; Everitt, 1968). These analyses have been found particularly useful in disturbed basins. Increased sediment load, channel widening, and channel bar development have been documented in East Tennessee basins affected by strip mining (Osterkamp et al., 1984; Bryan and Hupp, in press) and in West Tennessee basins affected by channel modification (Simon and Hupp, 1986).

The study area is in the Hatchie River and Obion River basins, West Tennessee, both tributary to the Mississippi River (Fig. 1). These rivers cut unconsolidated and erodible material, predominantly of Quaternary Age, composed of alluvium and loess (see Simon and Hupp, this volume). Specific study reaches are on Cub and Cane Creeks (tributaries to the Hatchie River), Rutherford Fork Obion River, and the Hatchie River mainstem; excepting the latter, all of the study reaches have been straightened and dredged within the last 25 years (see Simon and Hupp, this volume).

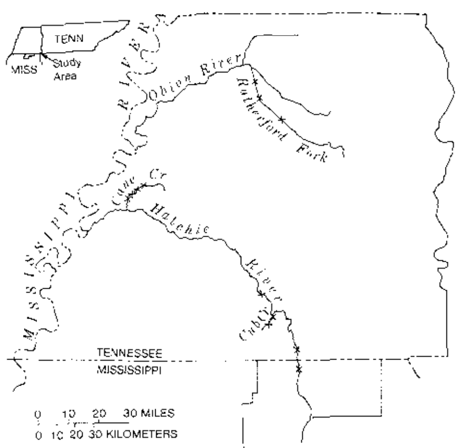

Figure 1-- Location of streams and study reaches in West Tennessee.

Simon (in press) has developed a six stage model of bed and bank adjustment following channel modification, summarized, by stages, as follows: I) the premodified state, with generally stable banks and mild aggradation, II) the constructed channel, usually trapezoidal in shape, III) bed lowering (degradation) in response to increased channel gradient, IV) the threshold stage where bed degradation continues beyond the critical height of the bank materials and mass wasting begins on banks, V) the aggradation stage, characterized by an increase in bed elevation, meandering thalweg, pronounced upper bank widening with considerable accretion on lower bank surfaces, and VI) continued aggradation, formation of alternate channel bars, and the resumption of relative stability on both bed and banks. Riparian vegetation reflects the stages of this model. In stage IV, for example, tilted trees and shrubs, slumped from the top bank, occur on the bank and allow dating of bank failure. During stage V, slumped trees still occur near the top bank. However, recently established woody plants grow on lower-bank surfaces and can be used for the determination of time since initial bank stability was achieved.

All of the reaches studied are located upstream of the area of maximum disturbance (AMD). This generally coincides with the upper limit of dredging and maximum degradation (Simon, in press). Downstream of the AMD, aggradation generally occurs from material associated with eroding bed and banks upstream (Simon in press; Simon and Hupp, this volume). All study reaches are in stage IV, V, or early VI of the Simon (in press) model except the Hatchie reaches, which are in stage I and serve as a control.

CHANNEL WIDENING

Channel widening, described in this report, refers to average increases in distance between top banks. Estimates of channel widening are made by determining ages of trees and shrubs that bear obvious deformations associated with bank slumping, then measuring the width of the slump block or the distance between the tilted stems and the present top-bank edge. Slump blocks of varying age, along a given reach, provide a history of recent bank failure. In some cases, bank widening is so rapid that all recent failures are one year or less old and previous slump blocks have been reworked by stream flow. An example of an actively widening reach is shown in Figure 2a and a generalized cross-section of variously aged slump blocks, tilted vegetation, and typical bank form is depicted in Figure 3.

Three study reaches, in the present study, are on the Rutherford Fork and the Hatchie River, four reaches on Cane Creek, and two reaches on Cub Creek (Fig. 1). Generally, a study reach includes bank areas that extend up to one kilometer above and below a bridge. Areas affected by bridge scour were avoided. A summary of the physical characteristics of each reach is provided in Table 1. Unstable banks typically have concave-upward bank profiles from low water elevation to top bank (Fig. 2a). Stabilizing banks become increasingly convex upward, particularly on midbank surfaces due to accretion and fluvial reworking.

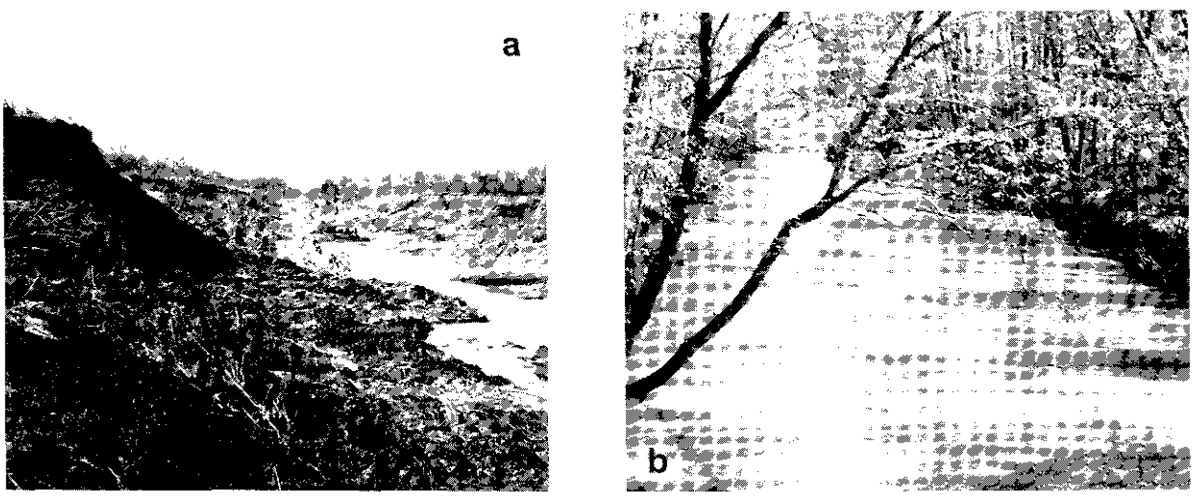

Figure 2-- (A) Cane Creek near Hunter Road, a highly degraded reach with unstable banks. (B) Hatchie River near Pocahontas TN., an unmodified reach.

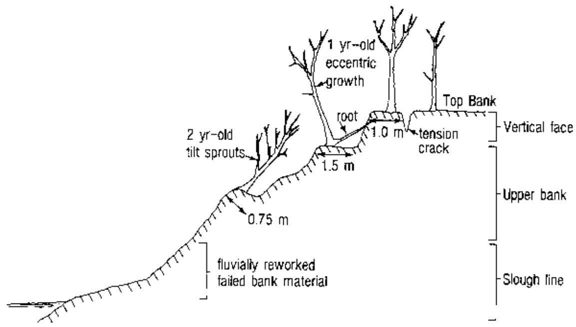

Figure 3-- Typical bank forms on modified streams with active widening. Note tree-ring evidence and slump block widths.

Figure 4-- Large slump block with with woody plants on Cane Creek.

Table 1. -- Summary of physical characteristics of study reaches.

	Stage	River km	Basin area (km^2)	Top bank width (m)	Date of modification	Bank profile
Rutherford Fork					1967	
Kenton	V	7.9	692	34.2		concave
State Rt. 105	V	16.7	616	35.7		concave
State Rt. 54	IV	27.7	523	32.3		concave
Cane Creek					1970	
Junius Lee	V	14.3	169	38.7		concave
Hunter Rd.	V	16.6	161	40.0		concave
Chs. Morris	V	20.3	130	32.0		concave
State Rt. 19	V	23.8	88	30.5		concave
Cub Creek					1970	
Hebron	(late) V	2.4	39	18.0		convex
County Rd.	IV	9.2	17	8.5		concave
Hatchie River						
Bolivar	I	217	3833			convex
Pocahontas	I	286	2168	33.2		convex
Walnut, MS	I	304	704	21.4		convex

Cane Creek has undergone the most degradation with present widening rates up to 2.4 meters per years (Simon and Hupp, 1986). Tension cracks and piping holes are common on top banks. Slump blocks up to 60 meters long form along the bank of all four reaches. These slumps form arcuate scallops along the bank that appear to migrate downstream with time. Failed material forms a hummocky slope below the vertical face (scarp) and is termed the upper bank (Fig. 3). The upper bank supports much tilted or otherwise damaged woody vegetation that moved downslope on slump blocks (Fig. 4). These trees (Fig. 4) allow the dating of the slump blocks on the basis of stem deformations associated with bank failure. Low-angle slides extend the upper bank downslope and fluvial reworking forms a slough line (Fig. 3).

Widening rates are greatest where incipient meanders are forming outside bends on the channel bed. It is inferred that, during high flows, the thalweg impinges on the banks, at these sites (scallops). Establishing vegetation is lacking on these scallops, whereas on apparently inside bends, the bank slopes support establishing young trees and shrubs, which suggests a greater degree of bank stability. These incipient meanders on straightened reaches enlarge in the downstream direction and account for a large measure of local variation in bank stability on all studied streams. Mass wasting occurs even on inside bends where the total bank height exceeds the critical bank height of the material (Simon and Hupp, this volume).

The Rutherford Fork study reaches are actively widening at an average rate of 0.75 meters per year on the lower two reaches and up to 1.5 meters per year along the upper reach (Fig. 1). Sets of parallel arcuate tension cracks are common on the top bank, particularly where the thalweg impinges upon the bank. As on Cane Creek, failure zones form large scallops along the top bank that appear to move downstream. Tree-ring data indicate annual bank failures on incipient outside bends, whereas incipient inside bends have failed periodically on about 2- to 3-year intervals. Small trees are beginning to establish on inside bends and are absent on the highly unstable outside bends.

Most bank profiles on the two more actively widening streams--Cane Creek and Rutherford Fork--are concave upward. Cub Creek, which has considerably more stable banks, exhibits convex upward bank profiles, except near the vertical face of the top bank and near the active-channel edge. Convex banks are more stable, have slow bank widening rates, and have an extensive zone of establishing vegetation. Active widening along the study reaches of Cub Creek (Fig. 1) is localized and generally occurs as small shallow slab failures along the vertical face. In some places a convex upper bank slope occurs to the top bank. Maximum widening rate on Cub Creek is from -0.10 meters to 0.20 meters per year. Incipient meanders have formed, however even on outside bends the energy of thalweg impingement upon the bank is apparently absorbed by accreting sediments and woody vegetation on the slough line and upper bank.

The Hatchie River study reaches are high in the Tennessee part of the basin and one is in northern Mississippi (Fig. 1). Although many of the tributaries to the Hatchie River have undergone channel modifications, the mainstem reflects relatively "natural" bank conditions (Fig. 2b). The most apparent differences between the Hatchie River and modified channels are the lack of a previously degraded channel bed and the presence of mature riparian vegetation down the bank to and into the low-water channel. The channel bed is rarely more than 3 meters below top bank (Fig. 2b), whereas the channel bed of Cane Creek (an order of magnitude smaller than the Hatchie) is often 12 meters below top bank (Fig. 2a). Banks of the Hatchie River are convex upward and exhibit no appreciable widening even on outside bends. A summary of botanical evidence for bank-widening rates, accretion rates and date of bank stability is provided in Table 2.

Table 2. -- Summary of botanical evidence of channel widening rates, bank accretion rates, and the site and timing of bank stability.

Stream	Channel widening, m/yr	Bank accretion, cm/yr	Date of initial stability	Site of initial stability
Cane Creek	2.4	7	1980	slough line
Rutherford Fork	0.75-1.5	5	1982	slough line
Cub Creek	0.1-0.2	5	1978	slough line
Hatchie River	0.01	0.5	----	------------

BANK ACCRETION

Generally, the slough line is the first to stabilize and revegetate, enhancing bank accretion (Fig. 3). Sediment carried by high water may deposit around the bases of riparian trees and shrubs. Parts of trees which remain buried for at least one growing season undergo changes such that subsequently formed wood is root tissue and adventitious roots sprout in what was once stem wood (Fig. 5). These responces to burial allow accurate determinations of date and rate of sediment deposition. Establishing trees and shrubs were analyzed for age and rate of sediment deposition above the initial root zone. A generalized buried sapling is shown in Figure 6.

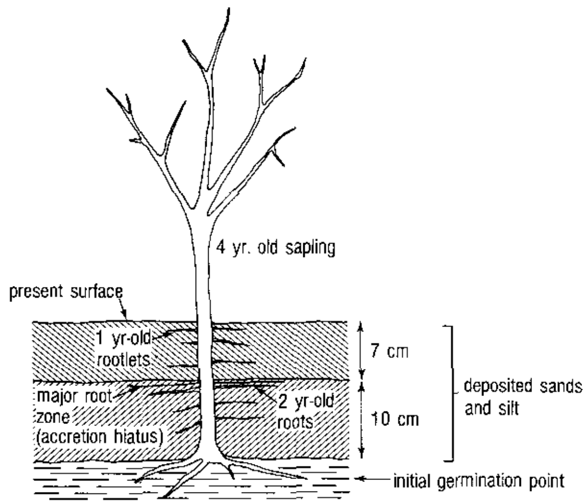

Figure 6-- Generalized form of buried sapling, showing dates and rates of sediment deposition.

Figure 5-- Photograph of partly buried boxelder, near Cub Creek. Note adventitious roots near top of 15 cm ruler, total burial depth is 32 cm.

The Cane Creek reaches have locally established vegetation on slough line surfaces (Fig. 3). This part of the bank also coincides with the area now subject to most bank accretion. Throughout the study reaches of Cane Creek 3- to 4-year old riparian saplings can be found on the slough line and correspond with the initiation of aggradation in about 1980 (Simon and Hupp, 1986). A strikingly consistent 20 to 30 cm of silt has been deposited over the initial root zones of the saplings, yielding a rate of bank accretion on the slough line about 7 cm/yr. The upper bank (Fig. 3) is still actively failing and has little established vegetation and no accreted sediments.

The Rutherford Fork has relatively little accreted bank sediments. However, the lowest study reach (Fig. 1) shows some deposition of sand on slough-line surfaces. Like Cane Creek, established woody plants, 1 to 3 years old, grow on the slough line, but the upper banks remain too unstable to support establishing woody vegetation. Along the lower study reach, these saplings have up to 10 cm of fine to medium sands deposited above the initial roots, yielding an average rate of about 5 cm/year. Upper reaches of Rutherford Fork are still in stage IV of the Simon (in press) model.

Cub Creek has had the greatest amount of bank accretion among the study streams, and is the most advanced (early stage VI) in terms of the Simon (in process) model. Along both study reaches (Fig. 1) 20 to 35 cm of deposited medium-sands are on the slough line and the upper-bank surfaces (Fig. 5). Established tree ages range from 3 to 7 years and indicate an average rate of deposition about 5 cm/year. Furthermore, vegetated bar areas form in the low-water channel and indicate stabilized conditions. These bars support shrub species (commonly alder, whose average age is 5 years) and bear evidence of mild aggradation (<5 cm) of coarse sand and pebbles.

The Hatchie River exhibits only mild accretion, principally on point bars and as thin layers of clays and silts on flood plain surfaces. Trees growing on banks are mature and usually more than 40 years old (Fig. 2b).

BANK FORM, STABILITY, AND SPECIES PRESENCE

Perhaps the most readily apparent indicator of unstable bank conditions is the general lack of establishing woody vegetation on bank slopes (Fig. 2a). Actively mass-wasting banks are unsuitable for establishing woody species. However, once some minimal degree of stability is attained, largely through geomorphic processes, riparian vegetation rapidly establishes. Once established, the woody plants substantially enhance bank stability and further ameliorate bank failure by slowing water velocities, which promotes increased sediment deposition and reduces low-angle slides through root mass development. Riparian vegetation, although unable to tolerate mass wasting, can survive relatively rapid accretion by sequentially producing new root zones (Fig. 6) along their stems (layering) into the newly deposited sediment (Sigafoos, 1964; Everitt, 1968; Bryan and Hupp, in press). Riparian vegetation (Table 3) forms a part of the suite of bottomland species that occur adjacent to most perennial streams and that can be quite different from species which typically occupy flood-plain areas (Hupp and Osterkamp, 1985).

Table 3. -- List of species growing on riparian surfaces, including only those that have germinated below the flood plain or top bank elevation. Species indicated with an asterisk are found on all four streams and are important early establishing species. Nomenclature follows Radford et al. (1968).

	Rutherford Fork	Cane Creek	Cub Creek	Hatchie River
**Acer negundo* (boxelder)	X	X	X	X
Acer saccharinum (silver maple)		X		X
Alnus serrulata (alder)		X	X	
Asimina triloba (paw paw)				X
**Betula nigra* (river birch)	X	X	X	X
Carpinus carolina (ironwood)			X	X
Carya cordiformis (bitternut hickory)				X
Celtis laevigata (sugarberry)				X
Cornus amomum (red willow)			X	
Fraxinus pennsylvanica (green ash)		X		X
Liquidambar styraciflua (sweet gum)			X	X
Liriodendron tulipifera (tulip tree)				X
Nyssa aquatica (tupelo)				X
**Platanus occidentalis* (sycamore)	X	X	X	X
Quercus lyrata (overcup oak)				X
Quercus phellos (willow oak)				X
Quercus nigra (water oak)				X
**Salix nigra* (black willow)	X	X	X	X
Taxodium distichum (bald cypress)				X
Ulmus alata (winged elm)			X	X
Ulmus rubra (slippery elm)			X	X
Total species	4	7	10	19

The banks of Rutherford Fork through Cane Creek to Cub Creek represent increasingly advanced forms of bank stability, following the stages outlined in the Simon (in press) model after channel modification. Generalized cross-sections of these streams, showing sediment accretion and establishing vegetation are shown in Figure 7.

Woody vegetation initially occupy surfaces of the upper slough line along all studied reaches. The principal "pioneer" species (willows, river birch, and boxelder) often establish as dense stands of seedlings. Presence of these species is the first indicator of stabilizing banks. Willow and river birch are particularly capable of surviving high rates of

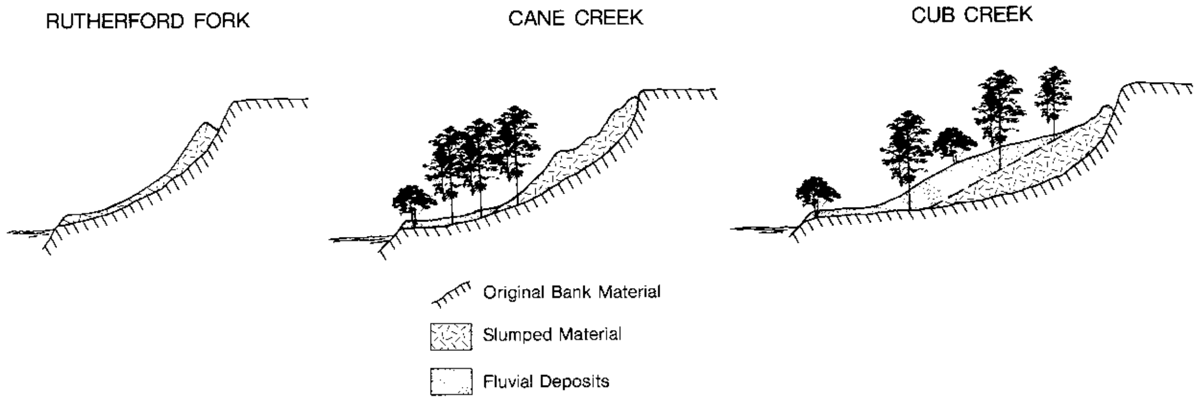

Figure 7-- Generalized cross sections of study reaches. Increasing stability occurs left to right. Note increasing convexity of mid-bank area and greater amounts of vegetation.

deposition. Several specimens, on other West Tennessee streams, have had up to 2 meters of deposition above the initial root zone. Along reaches, such as on Cub Creek, where sand is the predominant depositonal size class, alder and sycamore also are common. In addition to surviving high rates of deposition, riparian species also tolerate longer periods of inundation and flood damage than other bottomland species (Hupp and Osterkamp, 1985). It follows that the slough line, being the initial site of stability and accretion, is also the site of initial establishment of riparian vegetation.

A trend for increasing species richness with increasing site stability has been noted for many ecological systems. Inspection of Table 3 reveals a trend for increasing stability. The upper reaches of Rutherford Fork has only 4 species of establishing woody plants, Cane Creek has 7 species, and Cub Creek has 10 species; the upper reaches of the undisturbed Hatchie River have at least 19 species on its banks. Although this trend is presently a tentative argument, results support the increasing stability outlined in the Simon (in process) model of bank-slope adjustment.

SUMMARY

Woody riparian vegetation along modified stream channels provides an accurate tool for estimating dates and rates of bank failure, sediment accretion, and stabilizing bank conditions. Botanical evidence of bank-evolution processes is particularly useful where other types of information are lacking. Furthermore, establishment of vegetation (usually willow, river birch, boxelder, or sycamore) is an important indicator of initial-bank stability. Determination of the age of established plants allows for the estimation of the time necessary for banks to stabilize after channel modification (Table 2), and for pinpointing the areas on banks that are the first to stabilize (Table 2). Ultimately, botanical evidence of bank-slope adjustment should indicate the relative role failed-bank material plays in sediment transport following channel modification. Quantitative estimates of soil loss (from banks) and amount of accreted sediment stored on banks can be made when more reaches have been studied.

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EVOLUTION OF TWO DREDGED CHANNELS IN OKLAHOMA

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ABSTRACT

Eleven miles (18.3 km) of the Sugar Creek channel were dredged in 1967. Because the channel gradient, .0015 foot per foot, was too steep for the sandy alluvium, cross sectional area of the channel nearly doubled in two years. By 1969, maximum deposition on the lower end of Sugar Creek and in the Washita River was 6 feet, and the deposited volume was 650 acre-feet. By 1973, much of the deposition from Sugar Creek had been transported downstream, and the new bed was parallel to and approximately two feet higher than the bed of 1966.

To reduce the erosion taking place in the dredged channel Kellner jacks, riprap, tree plantings and fencing were installed. After several years, most of the dredged channel had stabilized. However, bank and bed erosion are still taking place further upstream as a result of the dredging. Within the Sugar Creek valley, the water table has been lowered and flooding has virtually been eliminated. Thus much of the former Bermuda grass pasture has been replaced by higher value crops.

The Tonkawa Creek channel was dredged for 3.6 miles (6 km) in 1973. This channel had 1.4 miles (2.3 km) of .0016 gradient and 2.2 miles (3.7 km) of .0007 gradient, and riprap was installed on the banks soon after dredging. Also, Tonkawa Creek has only 10 to 15 percent as much flow volume and peak as Sugar Creek. Thus there have been no erosional problems within the Tonkawa Creek channel.

INTRODUCTION

The Sugar Creek and Tonkawa Creek watersheds, tributaries of the Washita River in Southwestern Oklahoma, were authorized for flood prevention treatment by the Soil Conservation Service (SCS) under the Congressional Flood Control Act of 1944. The downstream reaches of these creeks were so choked with deposited sediment that storm flows normally overflowed onto adjacent land several times each year. Therefore, the Soil Conservation Service included channel dredging in the flood prevention plans for portions of both of these watersheds.

Both Tonkawa Creek and Sugar Creek flow through a fine sandy alluvium derived primarily from the Rush Springs sandstone. On Sugar Creek this fine sand material, the .0015 gradient, and the occasional large flow presented a stable channel design challenge. For Tonkawa Creek, however, which had less slope and infrequent small flows, a stable channel design was easily achieved.

Effectiveness of channel stabilization measures is analyzed and presented in this paper using nearly 20 years of data collected before and after dredging of these channels. Each of these streams had two gaging stations. However, the runoff records were discontinued a few years after the channels were dredged. Therefore, no analyses of sediment transport are included in this paper.

DESCRIPTION OF THE STUDY AREA

About two-thirds of both of the study watersheds are underlain by the Rush Springs sandstone. This sandstone is a reddish brown, very fine silty, highly crossbedded and loosely cemented formation. The watersheds lie primarily in the Cross Timbers land resource area characterized by a dense growth of scrub oak where the trees have not been removed.

The Tonkawa Creek and Sugar Creek watersheds contain 39 and 240 square miles (101 and 619 km^2) respectively. Land use of each watershed in 1974 was about 27 percent cultivation, 58 percent pasture and range, and 15 percent miscellaneous (USDA-ARS Southern Great Plains Research Watershed, 1983). Much of the land had been severely damaged by both sheet and gully erosion. The soils of these watersheds are dominantly sandy and range from silty clay loam to loamy sand and are permeable to freely permeable. The Sugar Creek alluvium is nearly all fine sand (10 percent is less than 62 microns, 60 percent is between 62 and 125 microns, and 30 percent is between 125 and 250 microns).

These watersheds lie in the subhumid climatic zone. Mean temperatures range from 38°F (3°C) in winter to 82°F (28°C) in summer. The average annual precipitation is about 28 inches (710 mm). Before Sugar Creek was dredged the channel capacity near Gracemont (figure 1) was only about 1000 cubic feet per second (cfs) or 28 cubic meters per second (cms) and decreased to only a few hundred cfs near the mouth. The SCS reported that there were 100 floods on Sugar Creek during the 27-year period from 1923 through 1949. Half or more of the flood plain was covered by 46 of these floods (SCS Work Plan Sugar Creek Watershed, 1959). Tonkawa Creek flowed through ox-bows of an abandoned channel of the Washita River and flooded adjacent farm land two to three times per year (Work Plan Tonkawa Creek Watershed, 1964).

CHANNEL DREDGING

Sugar Creek

The contract awarded late in 1966 for dredging of the lower 11 miles (17.7 km) of Sugar Creek specified .0015 slope, 45 feet (13.6 meters) bottom width, and 2.5:1 side slopes. However, after the first mile which began at the confluence with the Washita River was completed, the contractor elected to construct a pilot channel to drain the alluvium for the remaining 10 miles. Thus the dredging could be completed more economically. However, upon completion of the pilot channel a runoff event eroded the channel and eliminated the need for additional dredging.

In 1969 the dredging was extended about another four miles up Sugar Creek. The lower ends of White Bread and Keechi Creeks (figure 1) were also dredged. A drop structure was later installed on White Bread Creek just upstream from Highway 281.

Tonkawa Creek

In 1974 Tonkawa Creek (figure 1) was dredged for 3.6 miles (5.8 km) downstream from the gaging station. One drop structure was installed at the mouth of a tributary. The upper 1.4 miles (2.3 km) of dredged channel had 50 feet (15.1 meters) bottom width, 2.5:1 sides, and .0016 slope. The remainder of the channel had 60 feet (18.2 meters) bottom width, 3:1 sides, and .0007 slope. Rock riprap was placed on the channel sides for protection from flows up to five feet (1.5 meters) deep through most of the dredged reach.

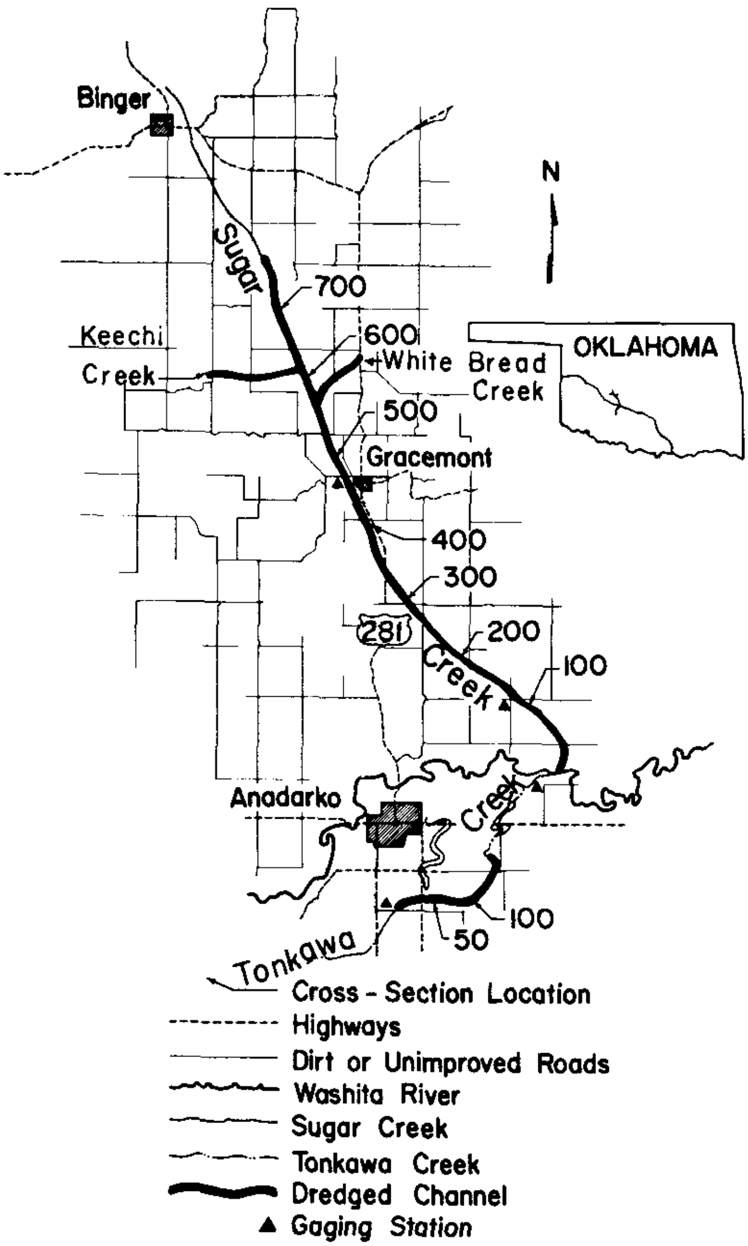


Figure 1. Sugar and Tonkawa Creek dredged channels

RESULTS AND DISCUSSION

Sugar Creek

After Sugar Creek was dredged, redeposition occurred in the lower end of the channel during runoff events that were concurrent with storm flow in the Washita River. This prompted the Soil Conservation Service to redredge the lower 1.5 miles (2.4 km) of channel in 1967 and the lower 0.5 mile (.8 km) of channel again in 1969. The sediment that was dredged near the mouth of Sugar Creek in 1969 removed the hump from the gradient shown for 1969 in figure 2. Deposition in the lower end of the channel during other years was removed by subsequent storm flows when the Washita River stage was low. In the Washita River sediment was deposited 6 feet (1.8 meter) deep for several miles below the Sugar Creek confluence before the Sugar Creek channel widened to a stable, non-eroding width. Over several years most of the 650 acre-feet (786,000 cubic meters) of sediment deposited in the Washita River were transported out of the affected reach.

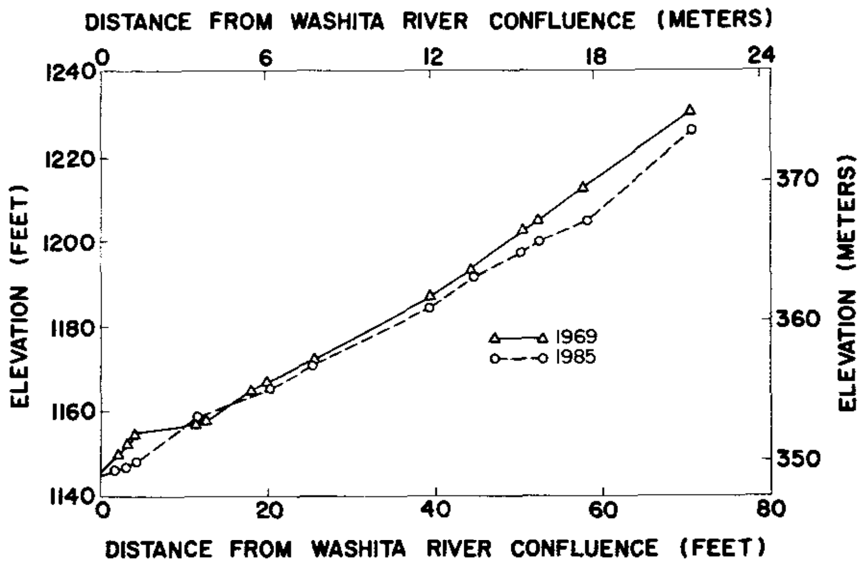


Figure 2. Sugar Creek channel gradient

A series of runoff events eroded the dredged pilot channel of Sugar Creek to excessive widths. Therefore, the Soil Conservation Service planned and contracted for channel protection. The protective measures installed in 1970 included 60,000 lineal feet (18,000 meters) of hog wire diversion and retard fencing, willow trees between the diversion fencing and the raw vertical banks, the rock riprap or Kellner jacks on the outside of bends. By 1973 these protective measures significantly reduced the width of the central reach of the channel (table 1). Most of the large increase in area of cross sections 20, 30, and 40 between 1969 and 1973 was caused by the redredging soon after the 1969 survey. The reduction in area at section 657 from 1969 to 1973 was caused by deposition of sediment eroded from upstream.

The resurvey of cross sections in 1985 showed that channel control measures installed from the mouth of Sugar Creek to Highway 281 have effectively reduced the channel width. Changes at section 190 within this reach are shown in figure 3. However, much of the 11 miles (18 km) of channel between Highway 281 and Binger has continued to widen and degrade. Although the protective measures were installed from the mouth to Gracemont, the willow trees did not become well established upstream from Highway 281. Extreme widening of the channel at section 414 (table 1) was initiated by pier failure under a pony truss bridge just upstream at the County road (figure 1). The collapsed

Table 1. Change in cross sectional area of Sugar Creek 1/

Section Location (Distance above confluence) (ft/100)	Area Change (ft^2)		
	1969-1973	1973-1985	Total
20	330	-698	-368
30	443	-570	-127
40	504	-600	-96
122	101	-401	-300
190	-35	-344	-379
241	-209	-220	-429
251	-100	-260	-360
260	-56	-266	-322
354	109	12	121
414	32	1846	1878
474	129	149	278
540	60	158	218
657	-51	1092	1041

1/ Negative numbers indicate deposition and positive numbers indicate erosion. To convert to square meters multiply square feet by .093.

bridge partially blocked the channel and diverted the flow. The top width at section 414 increased from 150 feet (45 meters) in 1973 to 300 feet (90 meters) in 1985. The continued degradation and widening of the dredged channel at section 657 in the upstream portion is shown in figure 4. Table 1 shows that the channel area at this section increased 1,092 ft^2 from 1973 to 1985.

The series of photographs in figure 5 show the Sugar Creek channel at about station 330 just downstream from Highway 281 in 1966 before dredging; 1967 soon after dredging of the pilot channel; 1970 just before installation of jacks, fences, and willow trees; and 1985 after willows had grown 15 to 20 feet (4.5 to 6 meters) above the 1985 ground level. The changes in channel cross section in these photos is probably about the same as that shown for section 354 in table 1.

Bergman and Sullivan (1963) reported that Sandstone Creek, a tributary of the Washita River in western Oklahoma changed from a rectangular to a V-shaped channel with less conveyance capacity following installation of floodwater retarding structures. Schoof, Thomas, and Boxley (1980) made a similar observation of the Winter Creek channel in south central Oklahoma following treatment with floodwater retarding structures. However, these drainage areas are much smaller than the Sugar Creek drainage area and the alluvium is probably more thoroughly graded. Therefore, we could not expect the same results.

The continuing degradation and widening of the Sugar Creek channel upstream from the dredged reach of channel supports the preliminary recommendation of Bowie (1982) based on findings in northern Mississippi. He said that for dredged channels subject to degradation, grade controls should be installed prior to or simultaneously with bank protection. The channel degradation of

Sugar Creek contributed to failure of several bridges.

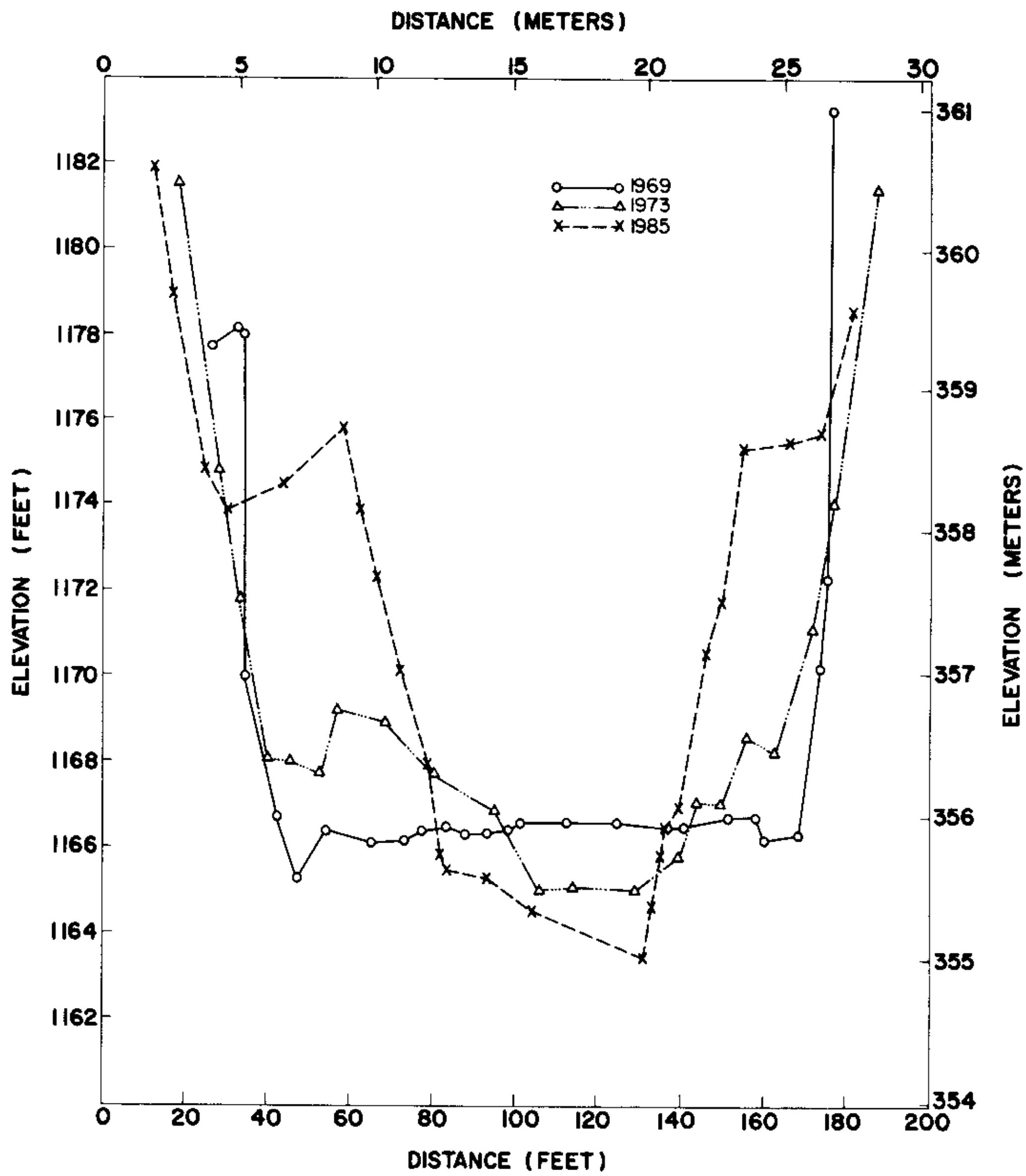


Figure 3. Sugar Creek cross-section 190 (19,000 ft. upstream from mouth).

The Kellner jacks on Sugar Creek have served well. Most of the jacks are covered or nearly covered with sediment. The Corps of Engineers (1962) stated that steel jacks can be considered a permanent type of protection. Existing installations were cited that had served up to 30 years. The hog wire toe fences will eventually rust out and fail. We do not know if the willow trees alone will prevent channel widening.

Dredging of Sugar Creek has lowered the water table and eliminated flooding along the dredged channel. Therefore, much of the former Bermuda grass

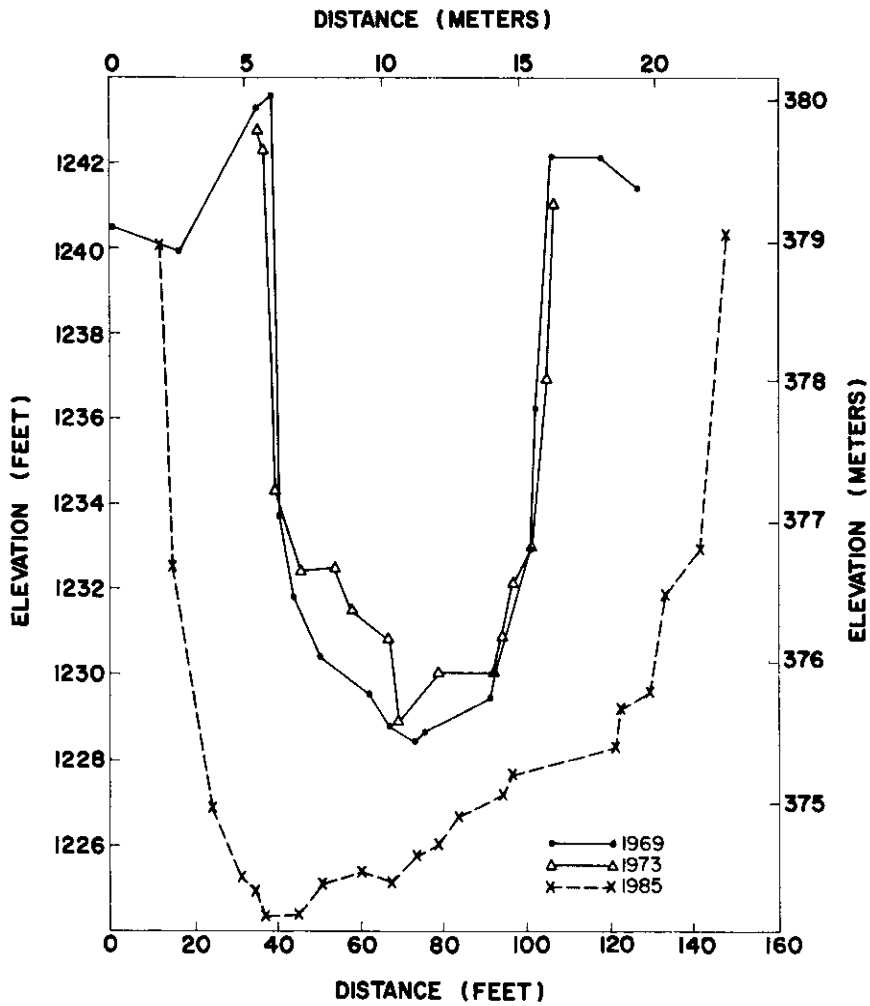
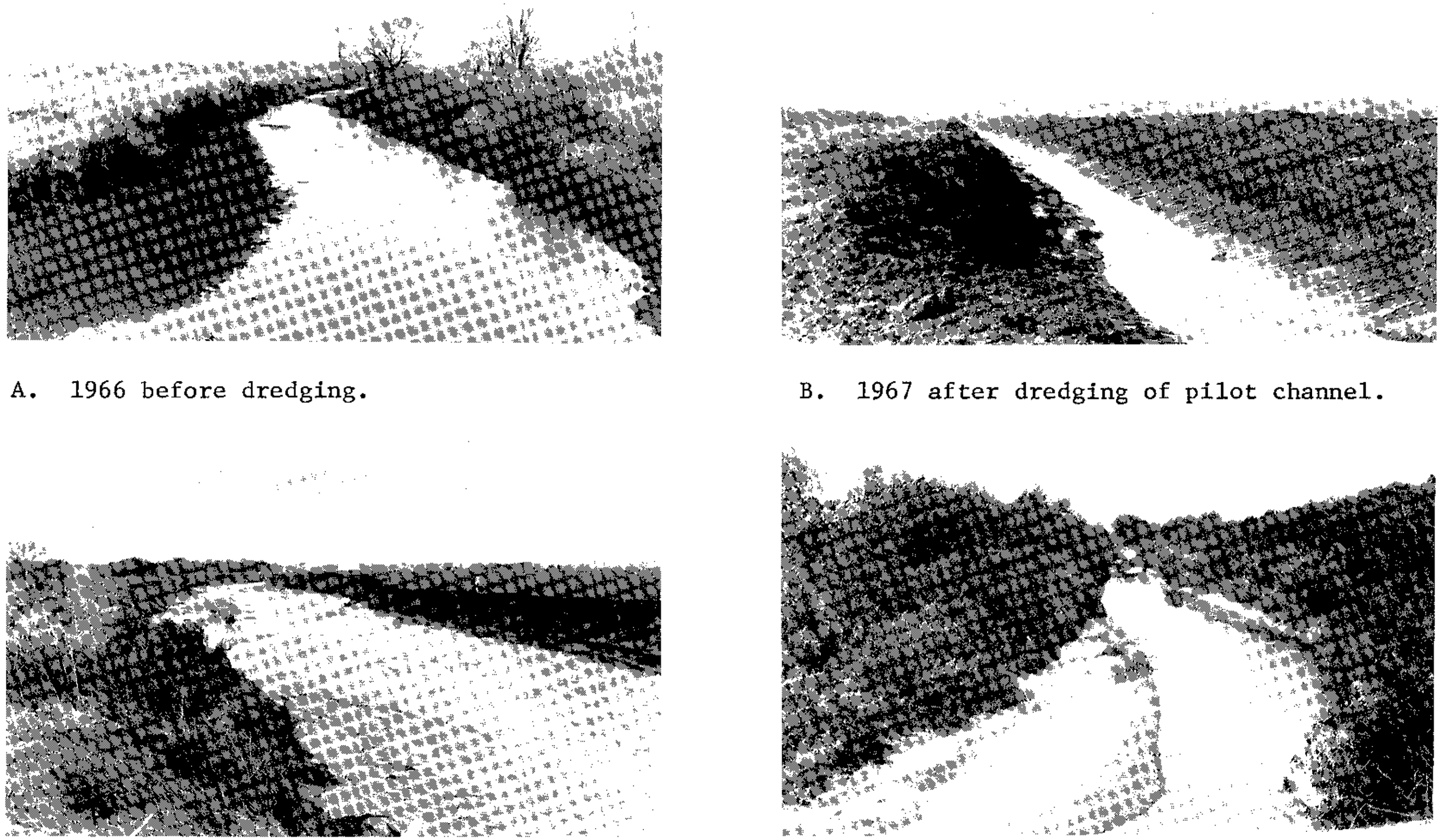


Figure 4. Sugar Creek cross-section 657 (65,000 ft upstream from mouth).

pasture has been replaced by higher value crops. However, in the future, construction of a multi-purpose reservoir on the main-stem of a creek should be considered as an alternative to channel dredging through fine, non-cohesive material.

Tonkawa Creek

The only significant change in Tonkawa Creek since dredging was aggradation which diminishes with distance downstream in the dredged channel as shown in Table 2. Most of this aggradation resulted from a washout in 1978 of the gaging station weir with a 3 feet (1 meter) overfall at the head of the dredged channel. The channel degraded upstream from the weir for more than 0.5 mile (.8 km). The dredged portion of Tonkawa Creek became overgrown with water grasses and sedges which slowed flow velocities and resulted in deposition of virtually all of the sandy sediment load. Changes at cross-section



A. 1966 before dredging.

B. 1967 after dredging of pilot channel.

C. 1970 before stabilization training treatment.

D. 1985 with willow trees 15-20 ft. tall.

Figure 5. Sugar Creek looking downstream near cross-section 330 (33,000 ft. upstream from mouth).

10 are shown in figure 6.

Table 2. Change in cross sectional area of Tonkawa Creek[1/]

Section Location (Distance downstream from gaging station) (ft/100)	Area Change (ft^2)		
	1974-1978	1978-1985	Total
2	-76	-227	-303
10	-79	-132	-211
30	-25	-60	-85
40	-22	-13	-35
70	-3	-14	-17
80	1	-16	-15
160	-11	-13	-24
170	2	-15	-13
180	0	-9	-9

1/ Negative numbers indicate deposition and positive numbers indicate erosion. To convert to square meters multiply square feet by .093.

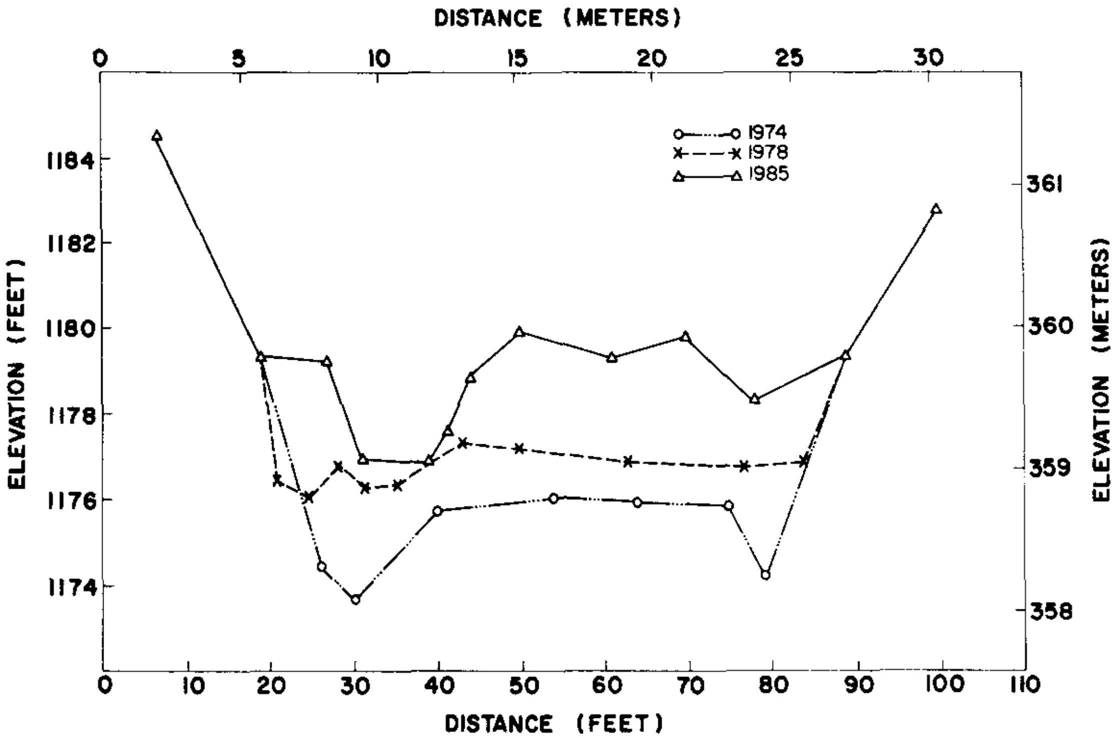


Figure 6. Tonkawa Creek cross-section 10 (1,000 ft. downstream from gaging station).

CONCLUSIONS

Severe channel degradation and widening is progressing upstream from the dredged reach of Sugar Creek. Erosion will probably continue and require additional protective measures in the future unless a drop structure is installed. Dredging and subsequent control of a channel in fine, non-cohesive material can be a very costly undertaking. Unforeseen expenses to control channel widening and degradation of a dredged channel and to repair and reconstruct bridges may ultimately create an unfavorable benefit-cost ratio. Therefore, construction of a multi-purpose reservoir on the main-stem of a creek should be considered as an alternative to channel dredging through fine, non-cohesive material.

Dredging of Tonkawa Creek has not created channel instability problems. However, flood flows probably exceeded 1000 cfs (28 cms) only once since the channel was dredged in 1974.

Dredging of Sugar Creek has lowered the water table and eliminated flooding along the dredged channel. Therefore, higher value crops can now be grown in place of the Bermuda grass pasture that existed before the channel was dredged.

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CHANNEL ADJUSTMENTS OF THE LOWER COLORADO RIVER

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ABSTRACT

The lower Colorado River experienced unusually high flows in 1983 and 1984. Peak discharges reached their highest levels since the completion of Hoover Dam in 1935. Sediment sampling, discharge measurements, cross-section surveys, and aerial photography were performed to monitor the behavior of the river.

Review of the data indicates that channel adjustments have occurred in several specific reaches. Comparison with historic data locates and quantifies these channel adjustments.

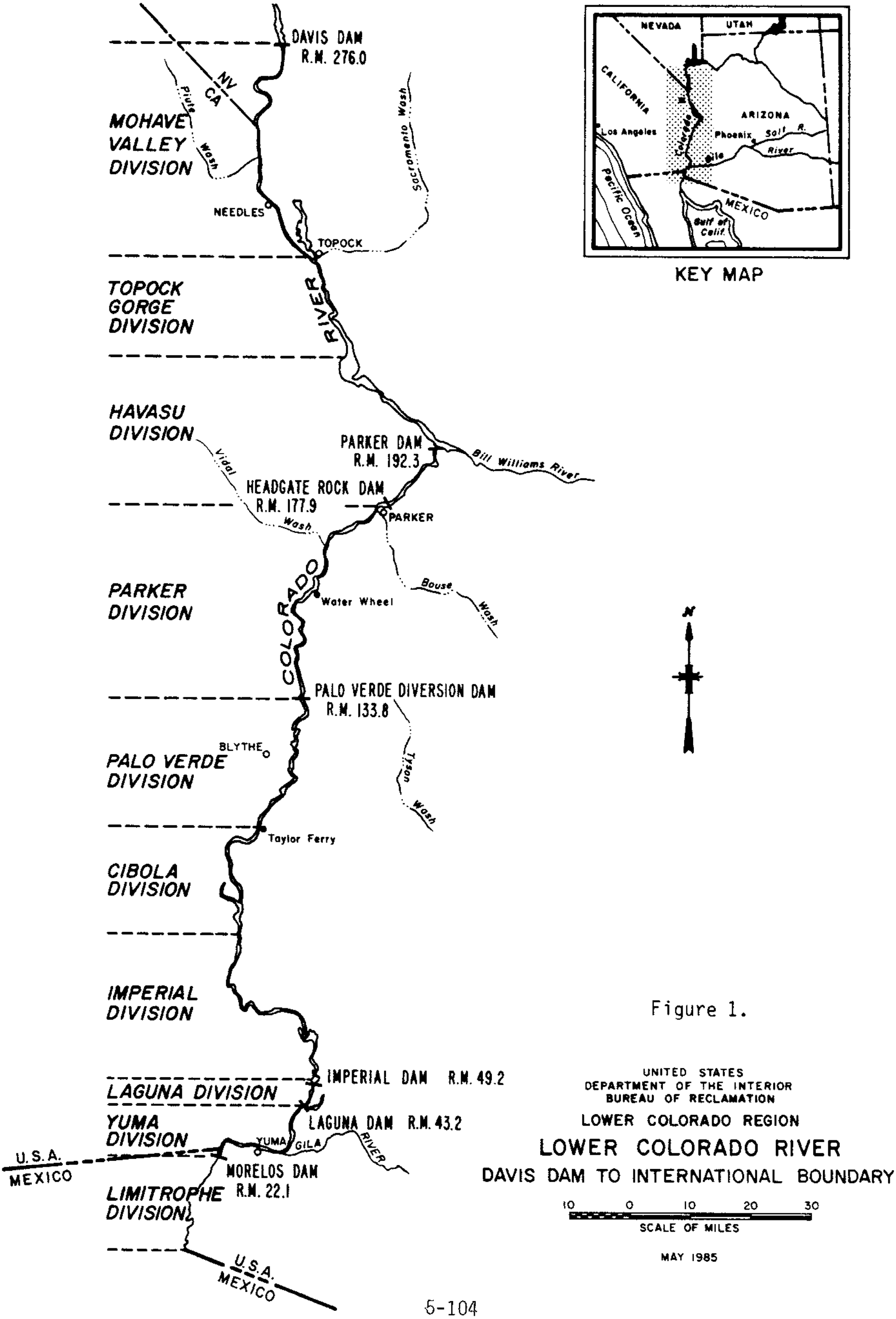
INTRODUCTION

The Colorado River originates in the high peaks of the Rocky Mountains in northern Colorado, traversing 1,400 miles of mountain valleys, canyonlands, and desert to its mouth in the Gulf of California. Its drainage area of 242,000 square miles includes portions of seven states. The lower Colorado River basin (Figure 1) is located in parts of Nevada, Arizona, and California. Here the river courses alternately through barren mountain canyons and broad alluvial valleys.

In its virgin state, the Colorado was subject to rampaging springtime floods as the mountain snowpack melted; during the autumn months the river receded to a relative trickle. It was one of the great sediment-bearing rivers of the world, transporting an average of 150 to 200 million tons of material annually.

The completion of Hoover Dam in 1935, and Davis and Parker Dams soon after, significantly altered the regime of the lower Colorado River. A marked reduction in flood peaks was the most notable change. Instead of the yearly cycle of severe spring floods followed by periods of low flow, spring runoff is stored in the reservoirs and released throughout the summer as needed downstream. The other major effect of the dam closures was interception of the river's sediment load. The sediment is deposited in the reservoirs and clear water discharged downstream. The remaining sediment sources are largely the riverbed and riverbanks, with some contribution from tributary desert washes and rivers during an occasional runoff event.

In addition to the reservoir storage and power generation effects of the dams, the natural flow of the lower Colorado River is affected by irrigation, municipal, and intermountain diversions, groundwater withdrawals, and return flows from irrigated areas.



DAVIS DAM
R.M. 276.0
NV
CA
Piute Wash
Sacramento Wash
MOHAVE VALLEY DIVISION
NEEDLES
TOPOCK
TOPOCK GORGE DIVISION
RIVER
HAVASU DIVISION
PARKER DAM
R.M. 192.3
Bill Williams River
Vidal Wash
HEADGATE ROCK DAM
R.M. 177.9
PARKER
Bouse Wash
PARKER DIVISION
Water Wheel
COLORADO
PALO VERDE DIVERSION DAM
R.M. 133.8
BLYTHE
Tyson Wash
PALO VERDE DIVISION
Taylor Ferry
CIBOLA DIVISION
IMPERIAL DIVISION
IMPERIAL DAM R.M. 49.2
LAGUNA DIVISION
LAGUNA DAM R.M. 43.2
YUMA DIVISION
YUMA
GILA
RIVER
U.S.A.
MEXICO
MORELOS DAM
R.M. 22.1
LIMITROPHE DIVISION
U.S.A.
MEXICO
N
NEVADA
UTAH
CALIFORNIA
ARIZONA
Los Angeles
Phoenix
Salt R.
Gila River
Colorado
Pacific Ocean
MEXICO
Gulf of Calif.
KEY MAP
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
LOWER COLORADO REGION
LOWER COLORADO RIVER
DAVIS DAM TO INTERNATIONAL BOUNDARY
10 0 10 20 30
SCALE OF MILES
MAY 1985

Figure 1.

Colorado River basin runoff for calendar year 1983 was the highest experienced since the completion of Hoover Dam, necessitating record releases from lower Colorado River dams beginning in June 1983. Runoff during 1984 was even greater, requiring continued high releases. Figure 2 illustrates historical discharges below Hoover Dam, including the large peaks before Hoover and the record releases in 1983. Although downstream diversions progressively reduce flows, the effect of record runoff on stations downriver is comparable to that below Hoover. The high flows have caused major channel adjustments and serious problems in some reaches of the river.

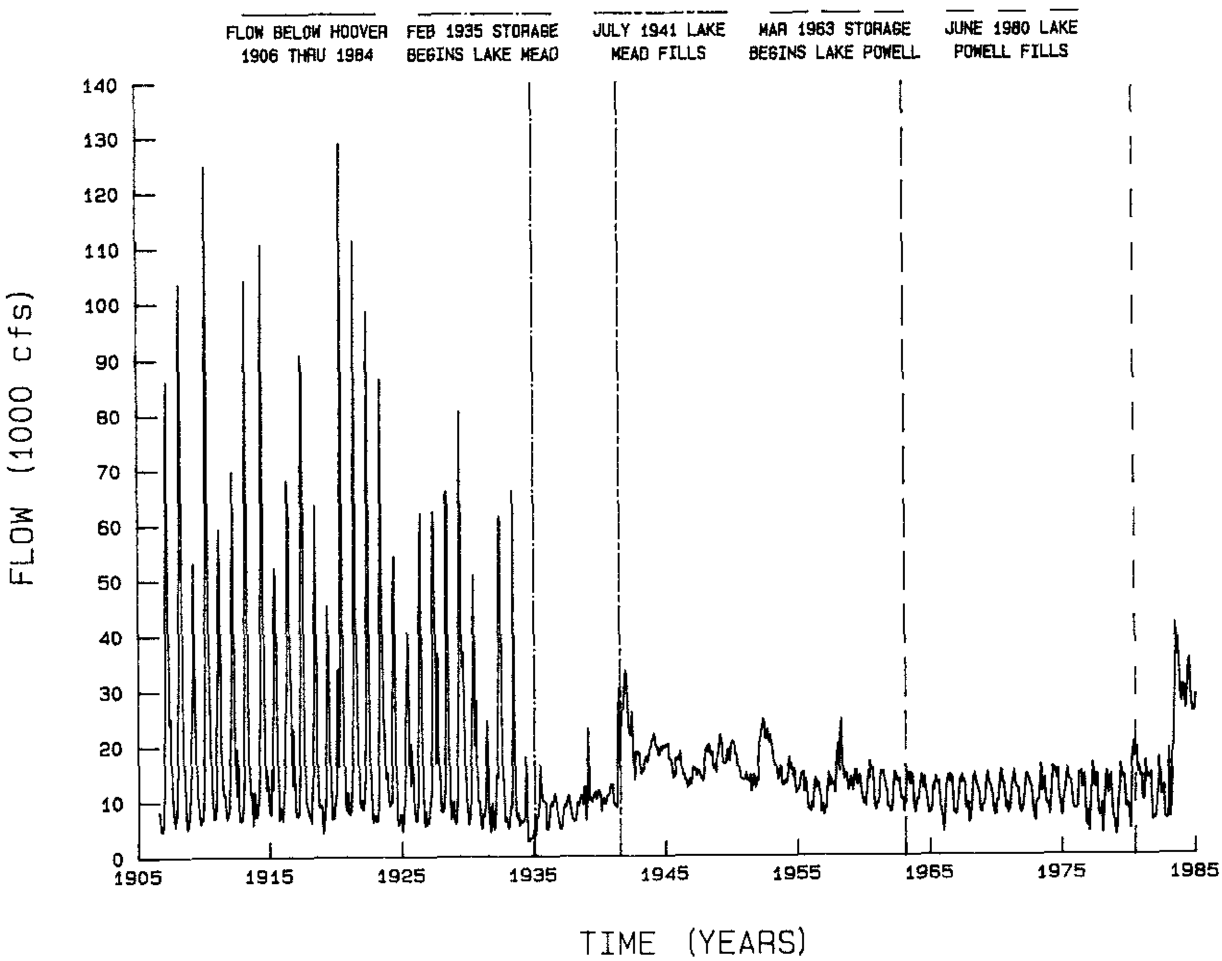


Figure 2. Historical discharges for Colorado River below Hoover Dam.

DESCRIPTION

The lower Colorado River is administratively divided into ten sections by geomorphic characteristics and/or hydraulic controls (Figure 1). River Miles are numbered from 0 at the Southerly International Boundary to 276 at Davis Dam. Major channelization projects began about 1945 on some reaches of the river, although levees were constructed near Yuma in the early 1900's.

The 66-mile section from Davis Dam upstream to Hoover Dam consists generally of inundated canyons and poses no river management problems.

In the Mohave Valley Division, the river flows eight miles through confining gravel banks, opening up to the five-mile-wide Mohave Valley, an old alluvial flood plain now utilized for agriculture. Existing channelization includes levees, riprap bank protection, jetties and training structures, and a settling basin maintained by dredging. Current construction is extending and armoring the levee system. At Topock, the river enters the scenic and environmentally sensitive Topock Gorge. No channelization has been completed in the Topock Gorge Division. Downstream, the Havasu Division consists of Lake Havasu, above Parker Dam, and a 15-mile canyon section below Parker Dam which has not required channelization.

The Parker Division extends into the Parker Valley, another alluvial valley approximately eight miles wide characterized by extensive meandering, cutoffs, and oxbows. Past corrective work in the upstream portion of the division involved dredging a channel, riprapping the bankline, and constructing training structures and a levee. The lower part of the Parker Division is experiencing significant lateral erosion, and planned work includes dry cuts to eliminate meanders, training structures, and riprap protection on the banklines. Farther south, the river flows through the continuous Palo Verde and Cibola Valleys, where major channelization has been performed. Riprap bank protection exists in both divisions, with training structures and backwater areas in the Palo Verde Division. In the Cibola Division, a dry cut has been excavated, levees constructed, and the bankline extensively riprapped.

The Imperial Division, with canyons and heavy vegetation, is largely inaccessible and no stabilization efforts have been made. The major feature of the Laguna Division is a dredge-maintained settling basin.

The Yuma Division encompasses a wide valley of delta deposits utilized by agriculture, with an extensive system of armored levees. Planned channelization work, partially completed, includes bankline protection and channel dredging. Downstream in the Limitrophe Division, the river forms the international boundary. An armored levee exists along the United States side, although the river channel itself is free to meander over the entire flood plain west of the levee. Major channel rectification is being planned for the entire division.

The slope of the lower Colorado River ranges from one to two feet per mile. Through the valley reaches, the upstream portions are generally degrading, with aggradation occurring in the downstream ends.

DATA AQUISITION

Data collection programs for the lower Colorado River include cross-section surveys, discharge measurements, sediment sampling, and aerial photography. Permanent river channel cross-section lines at about one mile intervals are surveyed approximately every 2 years. The Geological Survey and the Bureau of Reclamation maintain a number of rated and non-rated streamflow gauging stations on the lower Colorado River, making records available for riverflow data. Nine sediment sampling stations are currently used on the lower Colorado River. Suspended sediment is sampled twice monthly with a U. S. D-49 sampler; bed material is sampled twice yearly. Total sediment load is computed by the Modified Einstein method using sample data. Aerial photography of the river is performed semiannually.

Data acquisition continued during the high releases in 1983 and 1984, although some stations became inaccessible or were lost due to high water levels. Cross-section surveys and aerial photos were taken with greater than normal frequency, while sediment sampling in certain areas was somewhat curtailed.

SEDIMENT LOAD

Sediment has been one of the primary problems with management of the lower Colorado River. Prior to construction of the dams, the spring floods scoured the river channel annually, removing depositions made during the previous year. With the river controlled, the annual scouring no longer takes place and the sediment deposits tend to accumulate. Aggradation in the downstream sections of the valley reaches have periodically caused water levels to rise and necessitated channelization work. The settling basins in the Mohave and Laguna Divisions are designed to intercept sediment before downstream problems are caused. Intermittent dredging of the basins was necessary to remove the trapped material; however, dredging has not been required since the high flows began.

TABLE 1.

Lower Colorado River Sediment Load and Discharge Data

	Lower Colorado River near Topock		Lower Colorado River at Water Wheel	
Year	Total Sediment Load (tons)	Mean Discharge (cfs)	Total Sediment Load (tons)	Mean Discharge (cfs)
1970	579,000	10,700	1,178,000	7,990
1971	364,000	11,000	1,293,000	8,200
1972	405,000	10,700	786,000	8,100
1973	386,000	10,700	858,000	8,000
1974	508,000	11,000	1,115,000	8,940
1975	448,000	10,800	908,000	8,640
1976	593,000	10,100	612,000	7,790
1977	627,000	10,700	629,000	7,930
1978	613,000	10,000	662,000	8,250
1979	568,000	10,500	779,000	8,780
1980	779,000	14,700	1,645,000	13,800
1981	303,000	10,800	680,000	8,860
1982	263,000	10,300	358,000	7,290
1983	10,200,000	26,700	14,070,000	22,600

	Lower Colorado River at Taylor Ferry		Lower Colorado River below Yuma	
Year	Total Sediment Load (tons)	Mean Discharge (cfs)	Total Sediment Load (tons)	Mean Discharge (cfs)
1970	2,951,000	7,440	32,000	479
1971	2,424,000	7,780	40,000	511
1972	2,521,000	7,550	47,000	497
1973	2,321,000	7,510	125,000	876
1974	3,226,000	8,440	61,000	772
1975	3,045,000	8,460	66,000	843
1976	1,437,000	7,880	63,000	892
1977	1,429,000	7,630	69,000	829
1978	1,409,000	7,840	97,000	858
1979	2,281,000	8,370	437,000	2,330
1980	3,827,000	12,700	2,041,000	4,102
1981	1,688,000	8,300	270,000	960
1982	1,493,000	7,060	302,000	852
1983	14,680,000	21,200	4,024,000	13,018
1984			23,800,000	15,918

Table 1 lists recent sediment loads and corresponding water discharges at selected sampling stations. Sediment load has generally been stable or slightly decreasing in most reaches. The high riverflows beginning in June 1983 significantly reversed this trend, with enormous quantities of sediment transported by the high flows. For the sampling station near Topock, 1983 total load was 10.2 million tons as compared to a recent annual average of 0.5 million tons. At Water Wheel, 1983 sediment load reached 14.1 million tons versus an average 0.9 million tons; the load at Taylor Ferry was 14.7 versus 2.3 million tons. Below Yuma, the 1983 total load of 4 million tons far exceeded that for any previous year, while the 1984 sediment load was a staggering 23.8 million tons. Because of the sediment interception effect of the major dams, the origin of this material is the bed and banks of the river.

CHANNEL ADJUSTMENTS

One of the most dramatic changes in channel geometry is shown in Figure 3 for a cross section located at Yuma. At this point the Colorado River is bounded on each side by nonerodible conglomerate hills. Scour between old and new thalwegs is on the order of 25 feet here, with general degradation of eight to ten feet through the reach. Mean annual discharge at Yuma in 1983-1984 was approximately 1500 percent of recent normals, with a peak mean daily flow reaching 31,000 cfs in August 1983. Despite the channel degradation, the alluvial Yuma Valley was plagued by high groundwater levels during the highest flows.

Downstream from Yuma, the Limitrophe Division experienced appreciable lateral shifting of the channel and severe bankline erosion. Figure 4 shows a channel cross section near the southerly international boundary which is typical of the changes in the area. At this point the channel width increased by 800 feet, doubling the previous width, with a slight increase in depth. In some sections the lateral erosion was as much as 2000 feet. Figure 5 compares alignment of a channel section (a) during the rising leg of the high flows with (b) after sustained high flows. Lateral erosion on the order of 1000 feet has claimed extensive areas of farmland as the meander appears to be moving downstream. The meander could be cut off if high flows were to continue for a long period, as the river attempts to adjust to a new flow regime. Most of the erosion in the Limitrophe Division occurred on the Mexican side of the river; however, some emergency riprap placement was required on the United States bank. Peak 1983 mean daily flow in the Limitrophe Division was 33,000 cfs in August. Discharges in 1984 ranged from 10,000 to 20,000 cfs. Previous typical mean daily flows were less than 200 cfs, with peaks of perhaps a few thousand, and periods of no flow were common.

Another reach that sustained significant bankline erosion during the high discharges was the lower Parker Division, where the riverbanks reach heights of 30 feet. The sediment load observed at Water Wheel and Taylor Ferry is believed to be directly attributable to this erosion. An estimated 43 acres of riverbank was lost in 1983-84 through this 27-mile section. An example of the channel adjustment in the Parker Division is shown in Figure 6, where the channel has widened by at least 500 feet. Meanders appear to be developing more fully, with the outside bends eroding and deposition in the form of sand bars on the inside of the bends. Peak mean daily flow in the Parker Division approached 40,000 cfs in July 1983.

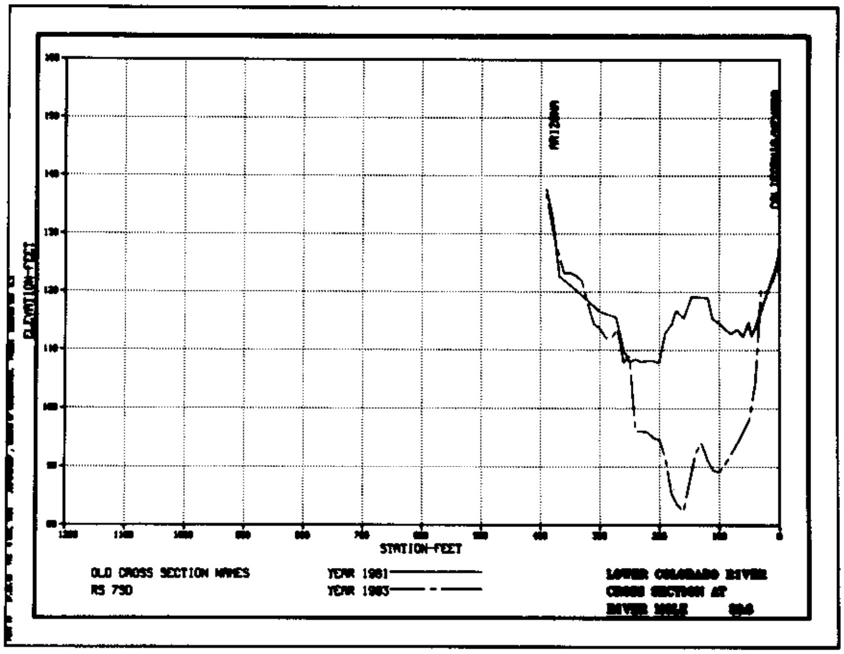


Figure 3. Channel cross-section at River Mile 30.6, Yuma Division.

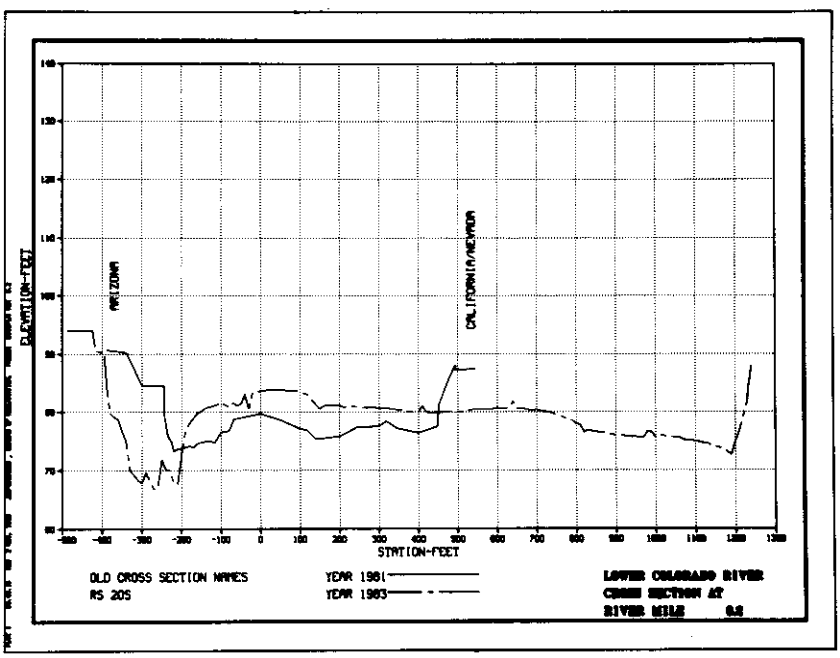
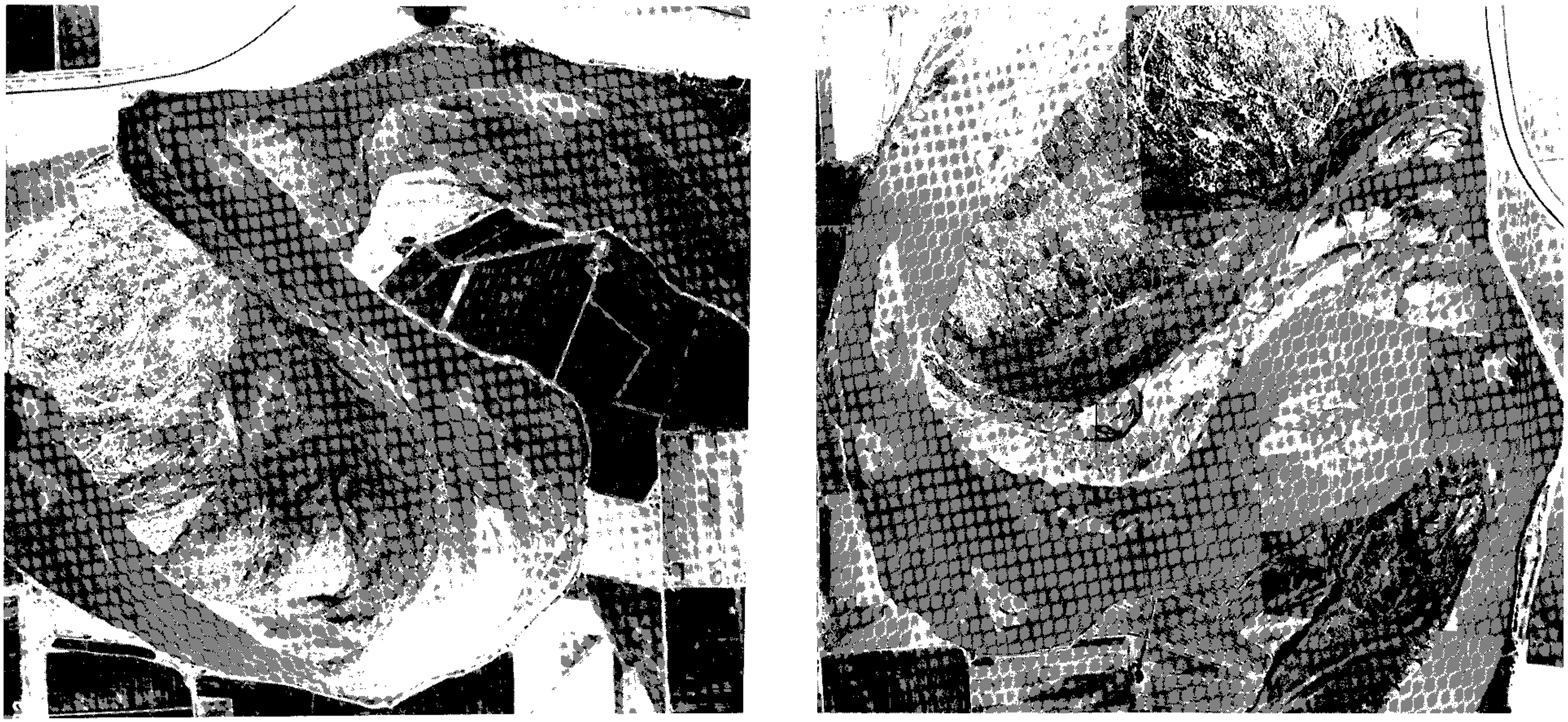


Figure 4. Channel Cross-section at River Mile 0.2, Limitrophe Division.



(a) (b)

Figure 5. Aerial view at River Mile 7.5, Limitrophe Division. 1 inch = approximately 1700 feet.
(a) June 22, 1983; Q = 12,500 cfs
(b) October 6, 1984; Q = 17,100 cfs

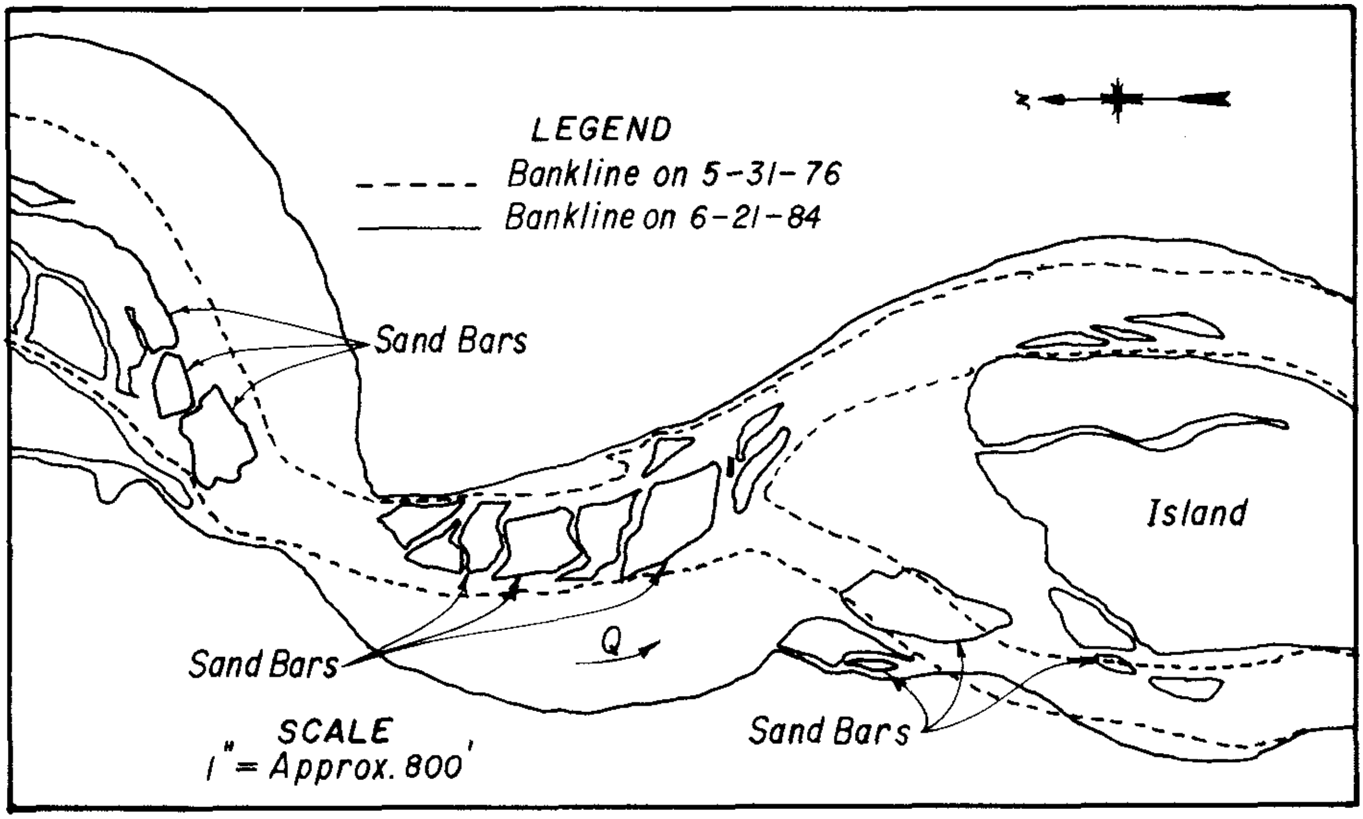


Figure 6. River channel realignment at River Mile 159, Parker Division.

The Palo Verde Division with its channelization was not broadly damaged, but isolated problems of bankline erosion did occur where large eddy currents developed at the onset of the high flows. These problem areas appeared and grew rapidly, and several emergency riprap placements were required.

The river channel through the Mohave Valley Division exhibits general scouring throughout the reach, but no lateral shifting. For example, at Topock, cross-section comparisons show a thalweg that has deepened by 15 feet. The large quantities of sediment transported through this section have deposited in the headwaters of Lake Havasu, at the downstream end of the Topock Gorge Division. The newly created delta is readily apparent in aerial photos. Based on cross-section data, a calculated 8,000,000 cubic yards of material has been deposited in the delta since the advent of the high releases. Mean daily discharge through the Mohave Valley peaked at 45,000 cfs in July 1983.

Ripples were the predominant bedform of the lower Colorado prior to 1983. Sediment concentration was usually less than 100 parts per million (ppm). During the high flows, large numbers of boils with high sediment concentrations were observed indicating a bed configuration of dunes. Exceptions are the channel sections below the dams which are naturally armored. Concentrations during peak flows of 1983 were generally larger than pre-1983. During 1984, sediment concentrations in the Mohave Valley declined appreciably while in the Limitrophe Division, concentrations ranged from 700 to above 1000 ppm. A sediment study in the Limitrophe Division indicates a bedform in the transition zone during 1984.

SUMMARY

In general, during the high flows of 1983-84, stabilized reaches of the lower Colorado River experienced channel degradation, while lateral erosion of the bankline characterized unstabilized sections. These effects are in keeping with regime theories of alluvial rivers which relate variables such as discharge, depth, and sediment properties.

Major channelization projects are planned for the Parker and Limitrophe Divisions, probably the two most unstable reaches of the river. Design analysis must consider impacts of the recent high flows as related to the channel configurations favored by the river.

With the filling of Lake Powell behind Glen Canyon Dam, a buffer for absorbing large Colorado River basin runoffs no longer exists. Thus, higher flows than those of the past 15 years may be expected in the lower Colorado, requiring a period of further adjustments of the river channel.

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ALLUVIAL TRENDS: NOTCHED RIVER CONTROL STRUCTURES

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ABSTRACT

A program to retain the floodway conveyance along the Missouri River through the use of notched river control structures was initiated in 1975. The purpose of the program, which included all of the Bank Stabilization and Navigation Project from Sioux City, Iowa, to the Mouth, was to alter depositional patterns behind river control structures by increasing the flow conveyance of the river without affecting the navigation channel design criteria. The original Missouri River Bank Stabilization and Navigation Project objectives were to prevent erosion along the river and construct a navigation channel alignment that would require a minimum of dredging for maintenance of controlling dimensions. These objectives were primarily achieved by constructing control structures to establish a channel location and width which would develop a stable thalweg.

Within the dike systems, however, aggradation developed naturally in the shallow water areas, causing a reduction in the total water surface area. Continued contraction of the flow-way has, to a large extent, eliminated much of the shallow water areas along the convex shoreline of the river flow-way. The notch program was intended to maintain or renew shallow water areas by providing a means for water to pass through and between rock structures, thus preventing sediment deposition downstream and landward of the structures.

Over 1300 notches have been constructed since 1975 by excavating a portion of existing stone-fill structures or by omitting repairs on small portions of damaged structures. Habitat enhancement evolved as an indirect benefit of the notches, and the program was expanded in 1982 to incorporate underwater reefs in conjunction with the notches. The underwater reef concept was included since such reefs not only have an impact on sediment deposition, but also afford fish protection, food, security, and a reference point by reducing light and providing a discontinuous substrate. Investigations have shown that the notch program may not substantially reverse the trend of reduced water surface area along a channelized river, but the program may be instrumental in minimizing further reduction of open water areas by preserving existing pools, chutes, islands, and sandbar areas found along the Missouri River.

INTRODUCTION

The Missouri River, one of the major rivers of the United States, is the principal river of the northern plains and a major tributary of the Mississippi River comprising 42 percent of the Mississippi River drainage basin. The Missouri River drains an area of approximately 530,000 square miles. This drainage area includes all or parts of ten states and contains approximately one-sixth of the total area of the contiguous United States. The Missouri River is classified as the third largest river in drainage area on the North American continent. Only the MacKenzie and Mississippi Rivers surpass it in terms of drainage size (Richardson and Christian, 1976). In addition, sediment discharge of the Missouri River ranks it as tenth of all the major rivers of the world (Todd, 1970).

The three principal headwater streams, the Jefferson, the Madison, and the Gallatin Rivers, combine near Three Forks, Montana, to form the Missouri River. With the Jefferson, the Beaverhead, and the Red Rock headwaters, the Missouri River is 2,714 miles long--the longest river in the United States.

Navigation and flood control were the primary purposes of the project as authorized in 1944 and remain the primary purposes today. However, over the past 40-plus years the philosophy of river control has grown to include other issues such as those involving environmental preservation. The broader scope of this philosophy addresses not only the maintenance of the structural integrity of the Missouri River Stabilization and Navigation Project, but also the harmonious relationship the project has with nature.

In consonance with this philosophy, a notch program was implemented on the Missouri River beginning in 1975. The project reach was from the Mouth of the Missouri River near St. Louis to Sioux City, Iowa (the 732.3 miles which constitute the entire navigation channel). The goals of the notch program were 1) to maintain the flood flow conveyance capability of the river channel; 2) to maintain and/or increase the amount of shallow water riverine habitat; and 3) to obtain sufficient field data to evaluate the navigation channel and bank stabilization features, with a view to similar application to other appropriate waterways throughout the nation (U. S. Army Corps of Engineers, 1982).

By June 1, 1980, a total of 1,306 notches had been constructed. Of this total, 344 notches were constructed in the river reach from Sioux City, Iowa, to Rulo, Nebraska, and 962 notches were constructed in the river reach from Rulo to the Mouth (U. S. Corps of Engineers, MRD Series, 1982). Although the notch program encompasses the entire navigation channel, the upper Missouri reach from Sioux City to Rulo will be emphasized in this paper due to the subsequent reef program implemented in 1982.

For purposes of the notch study, the project reach was divided into two study areas. The upstream reach, from Sioux City to Rulo, was evaluated by the Corps of Engineers-Omaha District, and the downstream reach, from Rulo to the Mouth, was evaluated by the Corps of Engineers-Kansas City District. The division of the study reach was not only logical from the standpoint of efficient evaluation, but also because it conformed to the boundaries of the individual engineer districts. In addition, the nature of the river itself required different notch design parameters in the two study reaches.

NOTCH DESIGN PARAMETERS

The design widths of notches constructed in the Omaha District study reach were limited to three--15 feet, 20 feet, and 30 feet. All design widths were correlated to the 1973 Construction Reference Plane (CRP). The CRP is an imaginary sloping plane, which is used for referencing structure elevations, and represents a profile corresponding to a discharge that is equalled or exceeded 75 percent of the time during the navigation season.

In addition to the design widths, the bank location of the notch structure, the placement location of the notch within the structure, and the depth of the notch were also important. Table 1 summarizes the number of existing notches as of June 1, 1980, by bank type and original design width, on the Missouri

River from Sioux City, Iowa, to Rulo, Nebraska (U. S. Corps of Engineers, 1982).

Table 1

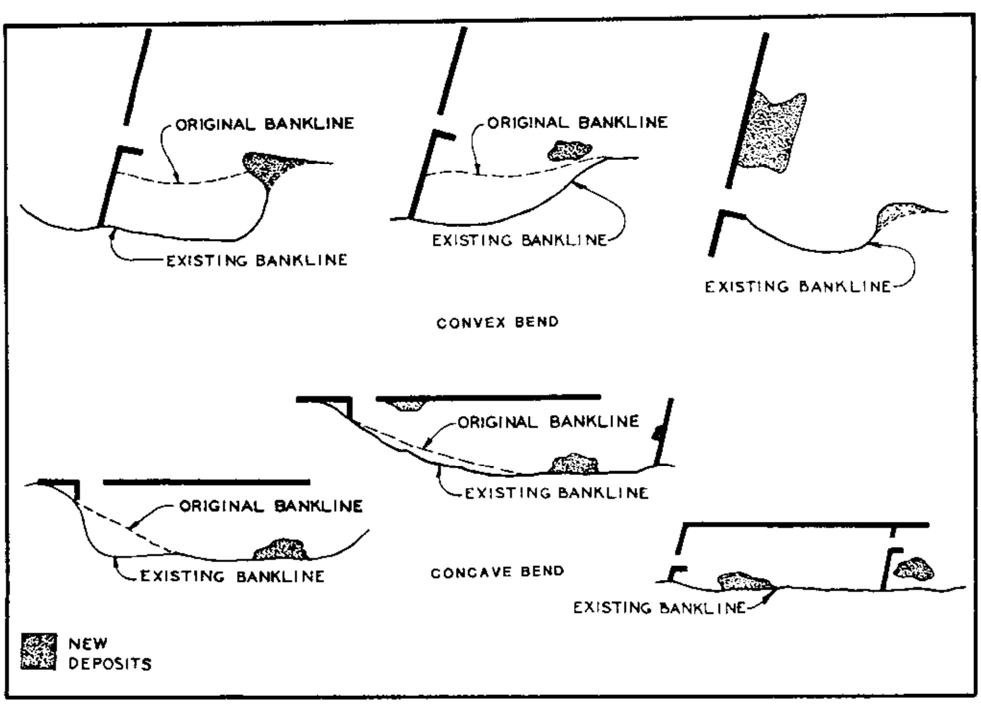
Notches Constructed on the Missouri River by Bank Type and Notch Design Width at CRP from Sioux City, Iowa, to Rulo, Nebraska

Bank Type	Notch Design Width at CRP 15 ft.	20 ft.	30 ft.	Total
Concave (outside of bend)	44	92	23	159
Convex (inside of bend)	19	146	20	185
Total	63	238	43	344

Structures located on the concave bank of each bend are revetment structures aligned parallel to the flow of the river. The convex bank was developed by the construction of dike structures designed perpendicular to the flow. The dike systems were originally designed to promote the deposition of sediments between structures which eventually would convert shallow water into sandbar and terrestrial habitat.

One of the purposes of the notch program was to provide a means for water to pass through and between the rock structures, thereby preventing sediment deposition downstream and landward of the structures to promote aquatic habitat. Depositional patterns for notched dikes and notched revetments along with original and existing banklines are shown in the figure below.

Depositional Patterns for
Notched Dike Structures and Notched Revetment Structures



In the downstream reach, from Rulo to the Mouth (a distance of 498 river miles), most of the structures had notch widths of 20 feet, 50 feet, or 100 feet. Wider construction widths, however, were used in the downstream 350 miles of the river because the dikes are generally longer than those found further upstream. Table 2 summarizes the number of existing notches as of June 1, 1980, by structure type and constructed notch width on the Missouri River in the Kansas City District from Rulo, Nebraska, to the Mouth. The difference between Table 1 and Table 2 is in the identification parameter used by the Omaha and Kansas City District offices (U. S. Corps of Engineers, 1982).

Table 2

Notches on the Missouri River by Structure Type and Construction Width from Rulo, Nebraska to the Mouth

Structure Type	Construction Width (ft.) at CRP							Total
	20	20	20-49	50	51-99	100	100	
Dike (D)	1	13	14	117	4	7	0	156
Revetment (R)	1	147	28	592	13	21	4	806
Total	2	160	42	709	17	28	4	962

DATA COLLECTION

Each district conducted data collection activities for the river reaches within its respective boundary. The Omaha District collected data on the physical conditions of over 150 notches during 1976, 1977, and 1979. Qualitative evaluations of debris retention, flow velocity, aquatic vegetation, and erosion were made during each inspection and quantitative velocity measurements were taken at selected notch locations. These data were then analyzed by making comparisons between the three inspection years, the three design widths, and the type of bank location.

In order to get detailed information on the condition of the bankline and water depths immediately downstream from the notches, hydrographic surveys were scheduled by the Omaha District over a three year period in the upstream reach. Forty-one surveys were completed, and six structures were surveyed during two different years which allowed for a comparative analysis. Of these six structures, three of the notches were in revetment structures and the remaining three were in dikes.

The Kansas City District selected 40 notched structures for detailed physical analysis from 1975 through 1979. The 40 notched structures were selected on the basis of representative samples for the lower 500 miles of the river. The notched structures were reviewed for similarities in bend location, flow attack angle, and vegetation.

Not only were engineering design evaluations made by the Omaha and Kansas City Districts, but a cooperative agreement was entered into with the National Stream Alteration Team (NSAT) of the U. S. Fish and Wildlife Service to study the impact of the notches on the aquatic environment of the areas downstream of the notches.

Environmental data, such as the number and species of fish, were collected and analyzed by study teams from the University of Missouri - Columbia, the University of Kansas, Iowa State University, and the Nebraska Game and Parks Commission. In 1977 and 1978, a total of 32 structures were studied in regard to aquatic habitat downstream and landward from notches. The structures were visited monthly during the period May to October. Fish sampling was accomplished using a trap, hoop net, seine, and shocker. Other essential data collected included date, time, surface water temperature, water clarity, water level, and current velocity. Visual observations on structure conditions and wildlife signs were also recorded.

NOTCH EVALUATIONS

The notch program evaluation with regard to sedimentation was primarily influenced by the following factors: debris, erosion, flow velocity and discharge. These factors were correlated to structure type and location. Flow velocity and discharge were also correlated to notch depth.

Debris

The main problem in the Omaha District with dike notches was the debris which had to be removed if the notches were to function as designed. As a point of note, and in accordance with Nomenclature for Hydraulics (ASCE, 1962), the Corps of Engineers defined debris in the notch study as "the amount of sediment, snags, garbage, and any other materials lodged in the notch, causing the flow to become restricted, not allowing the notch to function as designed" (U. S. Corps of Engineers, 1982).

Bank location appeared to be important in evaluating the impact of debris and it was found that the concave side had retained more open notches. Less than 20 percent of the notches on the concave side (revetment structures) had debris while 35 percent of the dike structures on the convex side had debris. It was also found that when notch widths were compared, the smallest, or 15-foot notches, were more likely to collect debris than the wider notches. The Omaha District found only a 10 percent difference but thought the difference to be significant enough when considering maintenance costs.

In contrast, the debris problem did not seem to be a pertinent issue in the downstream reach from Rulo to the Mouth--probably because the notch widths were generally larger than in the upstream study area. Some maintenance problems occurred in notches with 20-foot widths. The debris was limited in notches with 50-foot widths and did not seem to endanger the bankline with a high erosion rate. Notches larger than 50 feet appeared to present no problems with debris accumulation.

Erosion

The evaluation of erosion and its impact on sedimentation in the notch areas was limited in general to the configuration of the low overbank area downstream and landward of the notches. The figure presented earlier in this paper reveals the findings on alluvial trends. In the evaluation process the erosion was subjectively rated according to classifications of none, very light, light, moderate, and heavy. A heavy rating indicated erosion was severe and could develop into a problem if it began to flank the structure.

A low rating indicated mild or no erosion and could also mean some sediment deposition was occurring.

A large majority of the notches had light or very light erosion in 1976, but the percentage with moderate and heavy erosion increased during both 1977 and 1979. The "original" size of the notches did not appear to have a great effect on the amount of erosion, but the bank location of the notches revealed that convex banks have somewhat higher erosion than concave banks. The Kansas City District noted some problems with bank erosion where the notches were 100 feet wide. A small correlation existed between the ratings for flow and erosion; however, no indication was evident that heavy flow ratings resulted in higher amounts of erosion (U. S. Corps of Engineers, 1982).

Flow Velocity and Discharge

Comparisons of survey data on all the structures indicated a considerable variation in scour and deposition from one period to another. Due to the high degree of variability found in the Kansas City District study reach, it was difficult, if not impossible to detect any permanent trends or patterns. Thus, it seemed logical to conclude that the scour and deposition is greatly affected by the magnitude and duration of flow in the river and the location of the notch with respect to the main channel. The Kansas City District found that to develop more specific conclusions and recommendations, detailed surveys would be required at approximately one week intervals. Also, a more detailed comparison of scour and deposition with respect to magnitude, duration, and change in river stages would have to be performed.

Qualitative evaluations of flow velocity in the Omaha District reach indicated that the percentage of notches with light or moderate flow increased during the three inspection years. The discharge of the Missouri River was the same during the 1976 and 1977 surveys, but was 4,000 cfs higher in 1979. Even though the percentage of notches with light or moderate flows increased from 1976 to 1979, the increase was mostly in the 20-foot and larger size notches. It was found that a large number of the 15-foot wide notches had no flow.

Since 67 percent of the 15-foot wide notches are on the concave bank, the bank location may be directly responsible for the lower flow through the notches. The concave bank notches had discharges occurring over a wider range than those on the convex bank, and also had a greater diversity of flow conditions through the notches. The surveys of the notches in the revetment structures indicated above normal discharges promoted scour and low discharges caused some aggradation. When bank shapes were compared to velocity, most of the notches on the convex bank occurred in the middle ranges of 1 to 3 ft/sec (fps). The surveys of the notches in dikes showed that only small changes occurred in the water surface area and depth, even though the river discharges were varied. Data indicates that these notched dikes are maintaining the slack water habitats for which they were designed. The notches on the concave bends had more occurrences of velocities in the end ranges of less than 1 fps or more than 3 fps. The Omaha District felt that these velocity rates indicated the notches were providing desired slack water areas. More definitive information on sedimentation patterns in the Omaha District study reach was not available.

THE REEF PROGRAM

Originally, rock salvaged from the construction of notches was earmarked as a supply source for stone to minimize the cost of repairs to damaged river control structures. However, due to the age of the structures from which the stone was removed, the type of stone used in the original construction (limestone), and the weathering effects on that stone, it was speculated by the Omaha District that the stone in the structures identified for notching did not meet our current stone specifications for new construction or rehabilitation work. It was further suggested that the stone might better be used for aquatic habitat preservation in the form of reefs. Reefs not only have an impact on sediment deposition, but also afford fish protection, food, security, and a reference point by reducing light and providing a discontinuous substrate (Mahoney, 1982).

A total of twenty experimental reefs were authorized for construction with the stipulation that ten be located in the reach from Sioux City, Iowa, to Omaha, Nebraska and the remaining ten be constructed in the reach from Omaha, Nebraska to Rulo, Nebraska. The first ten notches with reefs were constructed by government forces in October, 1982, in the Omaha to Rulo reach of the Missouri River. The remaining ten experimental notches with reefs were constructed in May, 1983, in the reach from Sioux City to Omaha.

Dike structures were selected for notch-with-reef construction. All notches in this project were designed with 20-foot widths and the notch depths were 5 feet below the construction reference plane. Notch placement, however, varied within the dike structures. The reefs were each 25 feet long and were placed 50 feet downstream of the notch in each dike structure. The reefs were aligned with the notch in either a parallel or perpendicular manner.

DATA COLLECTION

Initial surveys were obtained at each dike location prior to construction of the notch and reef in order to assess existing water depths and bedform characteristics. Specifically, the area of each survey extended from 200 feet upstream of each proposed notched dike to the next dike downstream, and from the bankline to 200 feet riverward of the dike terminus. Particular attention was to be given to the area extending from the downstream side of each dike for a distance of 100 feet, and from the bankline to the riverward end of the dike. In these areas concentrated hydrographic survey data were obtained every 10 feet.

A second survey was performed after the construction was completed. The instructions for the second survey were the same as those given for the pre-construction survey, with one exception. The additional request was to locate the reef that was approximately 50 feet downstream from each notch.

The formulation of a comprehensive hydrographic survey technique was the first problem encountered in data collection. The first sets of survey data were obtained using established procedures employed in previous hydrographic survey projects. These procedures proved to be incompatible with the hydrographic surveys required by the reef project. The lack of control points in the survey technique only yielded data which proved to be inconsistent and incomparable with subsequent survey information.

It was readily apparent that the data collection and analyzing procedures lacked the sophistication for reliable comparison and interpretation. It was necessary to refine the survey technique for the reef project.

By the spring of 1984, a survey procedure had been developed whereby the survey data obtained was in consonance with the project requirements. The survey was based upon a grid system approach which used the dike structure as the control point. This proved to be successful and has yielded consistent, comparable data.

The second problem associated with data collection concerned the need for a reliable method of analyzing the available data ...especially from the first sets of survey data which were considered to be inconsistent. A reliable contour program was sought that would extrapolate the survey data in not only a 2-dimensional format, but also in a 3-dimensional format. The extrapolation of such data had to be reliable and it was learned that many such programs, especially where 3-dimensional contouring is involved, do not have a high degree of credible interpolation between contours. Finally, one such program called "Surface II" was located in December, 1984. The program was tested in June, 1985 using samples of the original survey data heretofore considered unusable due to its inconsistency and lack of control points. Surveys taken both before and immediately after construction were plotted and then compared with the grid survey method used in spring, 1985. The results have been impressive and very informative. For the first time the bedforms and structures can be graphically presented and studied in a format that is not purely numerical.

The third problem encountered during the study was high water in the Omaha to Rulo reach of the project area. In 1984 flood conditions prevailed and prevented surveys from being conducted in the spring. High discharge from the Platte River was the major factor contributing to the inundation of this river reach. Water overtopped the dike structures and made them inaccessible for survey purposes. The dike structures in the Sioux City to Omaha reach were not as affected by high water conditions due to the control from the mainstem dams. As a result of this environmental condition, surveys were unobtainable in the lower reach for a given period of time but obtainable in the upper reach for that same time period. Consequently, survey comparison of the two river reaches for the same time period were not always possible.

REEF EVALUATION

Based on historical development of data collection techniques from the early stages of this study, the current procedure for data collection is consistent and reliable. Early surveys, without a method of analytical interpretation, did not confirm the presence or condition of the reefs. Site inspections conducted in June, 1985 confirmed not only the presence of the reefs, but also the absence of sediment deposition on the reef structures. Confirmation was made by physically probing and/or sounding at reef locations. Additional confirmation was obtained by using the "Surface II" contour program as described previously. Evaluating the bedform data from all the reef areas is still being conducted using the "Surface II" contour program, but the analyses thus far completed, along with the June, 1985 site inspections, clearly demonstrate the reefs are in place and functioning as designed with little or no sedimentation occurring on the reef structures. Analysis of the contour maps showed the bedform characteristics after reef placement to be much more diverse than before construction. A strong contrast in sediment patterns was apparent. The reefs appear to greatly affect the sediment pattern and seem to eliminate the uniform riverbed depicted by survey data obtained before reef construction.

GENERAL CONCLUSIONS

Observations during the notch program produced results which should be beneficial to agencies considering such structures on other river systems. It is important to note, the notch program has been successful in accomplishing the project goals without adverse effects upon the navigation channel. Results of the notch study generally indicated the most effective notches were those which created relatively large areas of quiet water over the widest possible range of discharges without promoting sediment deposition. Both the qualitative and quantitative evaluations indicated more dike notches attained the discharges required to prohibit sediment deposition. Only small changes occurred in the water surface area and depth downstream of notched dike structures, even though the river discharges were varied.

In a notch construction program, the structure location being considered for notching needs to be assessed in terms of: 1) shape of the river bend; and 2) angle of the structure to the flow. In the order of importance, notch design parameters themselves were determined to be notch depth, notch width, bank location, and notch placement within the structure.

The minimum limit of the width and depth dimensions should allow enough flow to avoid excessive sediment and debris accumulation. The maximum size should not induce damaging erosion. On the Missouri River, for example, depths of 1 to 2 feet below CRP were used for the upstream reach where the discharge is mainly controlled by the reservoir system releases. Depths of 2 to 3 feet below CRP were used for the downstream portion where the tributaries have a greater impact on stage fluctuations. Although these dimensions worked the best under the given river conditions, it was determined that notches should be built to a variety of depths in order to provide a diversity of quiet water areas under differing river flow conditions.

The most effective notch widths used in the Missouri River notch program ranged from 20 to 50 feet. It was determined that notches which are equal to 20 feet but not greater than 50 feet wide have an appreciable effect in not only discouraging further accretion downstream and landward of the structure, but debris accumulation as well. In the final analysis, it was concluded that 1) the width of a notch in a <u>dike</u> should be 10-25% of the riverward length of the structure; and 2) the <u>width</u> and depth of the revetment notches should be 25-50% greater than the dike notches (U. S. Corps of Engineers, 1982).

It appears that bank location is also an important consideration. The main problem with the dike notches (convex bend locations) was the debris accumulation. The smallest, or 15-foot wide, notches appeared to be more susceptible to debris and had the largest percentage showing no flow. There were fewer debris problems with notches in revetment structures (concave bend locations). Overall, the notches in structures along concave bends did not perform as well as notches in dike structures located on convex bends even though convex banks experience somewhat higher erosion than concave banks.

The complementary reef program has clearly demonstrated the impact of the reef structures on sedimentation patterns. The reefs seem to eliminate the uniform riverbed characteristics depicted by survey data prior to reef construction. Field investigations have confirmed the reefs are in place and functioning as designed with little or no sedimentation occurring on the reef structures themselves.

Investigations clearly demonstrate that the notch program may not substantially reverse the trend of reduced water surface area along a channelized river, but may be instrumental in preserving existing open water areas for both the conveyance of flood flows and habitat diversity by preventing the complete elimination of pools, chutes, and sandbar areas found along the Missouri River.

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CHANNEL ADJUSTMENTS IN A YAZOO BLUFFLINE TRIBUTARY

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ABSTRACT

Massive upland erosion in north-central Mississippi during the late nineteenth and early twentieth centuries fostered major conservation projects, including reforestation, in this area. These efforts significantly reduced the upland sheet and gully erosion but, by the 1960's, a new problem of channel deterioration was beginning to occur. The continuing erosion of these channels has become a serious problem in itself as it affects the maintenance of flood capacity of downstream rivers. To quantify the scope of this erosion problem and to achieve a better understanding of the nature of channel adjustments, a 3.86 km reach of Goodwin Creek, a Yazoo River bluffline tributary, has been instrumented and repeatedly surveyed since 1977. Channel adjustments along this reach are the subject of this report.

The magnitudes of channel adjustments for this period of study were related to bed and bank material lithology whereas shorter-term or event-induced changes were variable in time and space. Banks composed of early-Holocene deposits were relatively stable except when undercut by bank toe removal, producing gravity failures of polygonal blocks along seams of relatively weak material. Late-Holocene deposits were more erodible and gravity-induced slab-failures occurred after bank toe removal, bank loading to or approaching worst case conditions, and tension crack formation. Although channel adjustments varied considerably by site and by study interval, the net erosion for a 2.81 km reach averaged 7,500 m^3 per year, or about 2.68 cubic meters per meter of channel length per year. This is equivalent to an annual erosion rate of about 5.5 metric tons per hectare (2.4 tons/acre) for the contributing watershed. However, individual storms occasionally produced net changes as large as 30 t/m at localized sites.

INTRODUCTION

The USDA Sedimentation Laboratory in 1977 entered into a four year cooperative study with the U. S. Army Corps of Engineers as part of the Section 32 Streambank Erosion Control and Demonstration Project. Goodwin Creek Experimental Watershed, 21.5 km^2 in area, was instrumented with 14 supercritical flumes for measuring water and sediment discharge, and a detailed channel survey program was initiated. The USDA Sedimentation Laboratory has continued this project as part of its ongoing research. The channel adjustments documented since 1977 are the subject of this report.

STUDY AREA

The general study area has been described in earlier publications (Vestal, 1956; Grissinger & Murphey, 1982, 1983; and Grissinger, et al., 1981, 1982). To briefly summarize, the watershed is situated on the eastern side of the Bluff or Loess Hills physiographic subprovince (Fig. 1). Watershed area is fairly evenly divided among row-crops, pasture, and woodlands. Most of the row crops (cotton, soybeans, grain sorghum) are now planted only in the flat valleys although some soybeans and grain sorghum are occasionally planted on the larger undissected upland areas. Most of the latter is in pasture while woodland occupies the often severely eroded valley sideslopes.

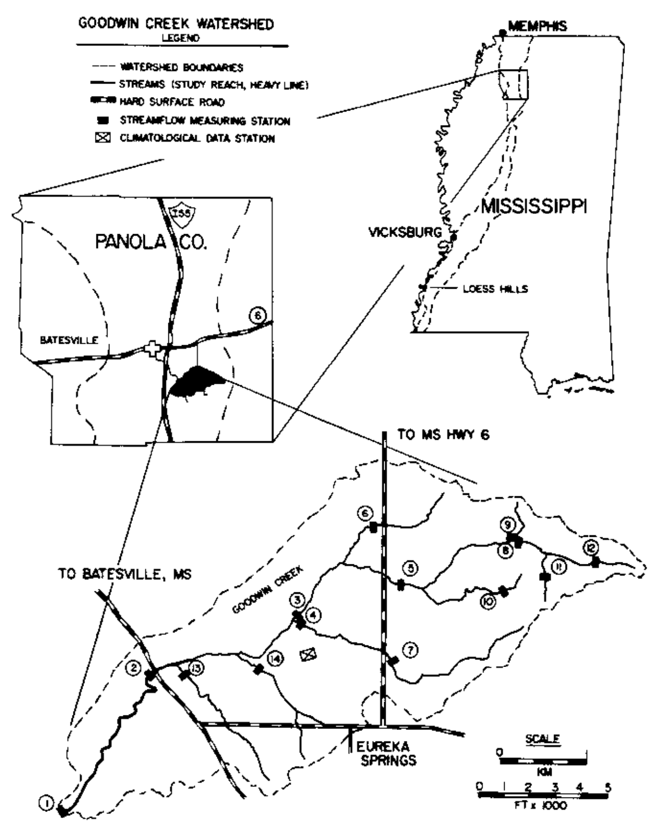

Fig. 1. Goodwin Creek Experimental Watershed, Panola County, Mississippi. The channel length included in this study extends upstream to just above flume 2.

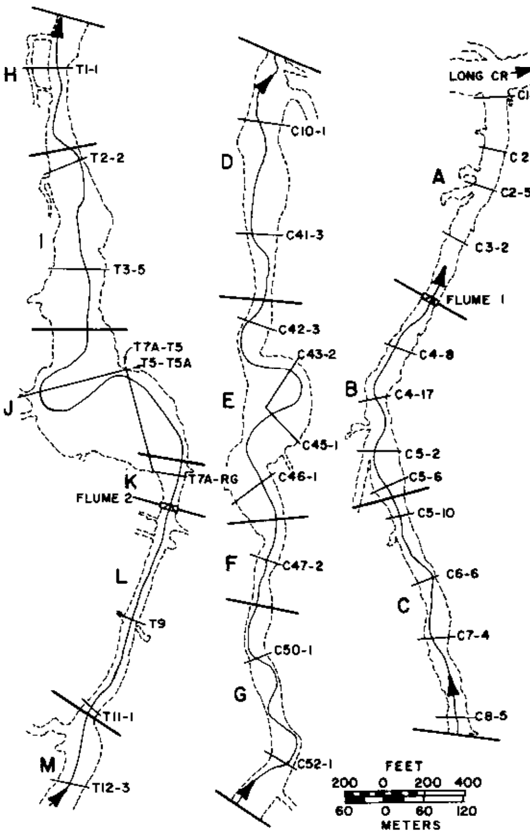

Fig. 2. Plan map of the 3.86 km study length of Goodwin Creek showing geomorphic type zones (letters), cross sections, and flume locations. Flow direction is from M to A. Thalweg and top bank positions are shown by the solid and dashed lines respectively.

In the upper reaches of the watershed, streamflow only occurs immediately following precipitation. In the lower reaches, a slight base flow usually persists for most of the year. Storm flows carry heavy loads of coarse sediment derived from upland tributaries and gullies and from the valley alluvium. The stream channels are greatly enlarged in places, are highly variable in form and are, almost without exception, deeply incised.

The uplands are draped with a mantle of Pleistocene loess which overlies the Citronelle deposits of sand and gravel. Alluvial deposits in the valley record three previous major periods of channel incision, the first prior to 13,000 years Before Present (yr BP), the second around 6,000 yr BP, and the third from about 3,000 to 200 yr BP (Grissinger and Murphey, 1983; Grissinger, et al., 1982). Identifiable lithologic units overlying each of these erosion surfaces are, in order of decreasing age, (a) an early-Holocene sequence of channel lag, organic bog, and gray silt overlain by a massive silt deposit with distinctive polygonal seams that act as planes of weakness (b) a mid-Holocene deposit of well-bedded relatively coarse-textured channel fill occupying a somewhat narrow but deep channel incision; and (c) a late-Holocene deposit of fine to coarse meander-belt alluvium. Superimposed over these three deposits are massive amounts of finely-layered silt and fine sand that were washed from the hills following European settlement.

PROCEDURES

Detailed 0.61 m (2 ft.) contour interval topographic surveys of all the main channel and primary tributaries of Goodwin Creek were prepared by the U. S. Army Corps of Engineers in 1977. This topography was constructed from a dense system of monumented cross-sections. The 3.86 km length of Goodwin Creek identified in Fig. 2 was selected for continuous resurveys on the basis of (1) the availability of suitable discharge data, (2) the absence of large tributaries, and (3) the range of channel forms. Based on these observed forms, this length of channel was sequentially divided into zones that appeared to be similar in shape, habit and/or vegetative cover (Murphey and Grissinger, 1983). These zones are delineated in Fig. 2 and are identified and characterized as: A - narrow, straight, Kudzu vine-covered banks; B - narrow to wide, slightly sinuous, half-vegetated, alternate bars; C - narrow, slightly sinuous, vegetated, alternate bars; D - wider, slightly sinuous, mostly unvegetated; E - 3 long wide bends, large point bars, unvegetated; F - narrow, straight, vegetated banks; G - 4 short narrow bends, high alternate bars, unvegetated; H - narrow, straight, vegetated; I - wide, straight, mid-channel bars; J - 2 long wide bends, large point bars, unvegetated, steeper channel slope; K - narrow, straight, rip-rapped banks; L - narrow, straight, vegetated; and M - wide, nearly straight, half-vegetated reaches.

Thirty of the 1977 cross sections were resurveyed in November, 1982. These cross sections, identified in Fig. 2, have been periodically resurveyed since 1982 to keep track of the adjustments. As of March, 1984, the date of the latest hydraulic data fully processed, seven resurveys were completed. Volume changes for these survey intervals are presented in Table 1 by cross section, in sequential order upstream from the confluence with Long Creek. Reach volume changes were calculated by multiplying the net area differences at each cross section by the length of channel halfway upstream and downstream to the adjacent cross sections. For this report, bed and bank material changes in volume were not separated. Although some local thalweg lowering was observed between 11/11/77 and 11/18/82, most of the net scour resulted from bank failure.

RESULTS AND DISCUSSION

The net scour or fill volumes for the thirty reaches comprising the 3.86 km study channel are presented in Table 1 for the seven survey intervals from 11-11-77 to 3-22-84. Discharge data are incomplete for the first interval, from 11-11-77 to 11-18-82, but are complete for the six subsequent intervals.

These data (Table 1) show two general trends; (1) scour dominates long-term (6.36 year) channel adjustment and (2) short-term changes are variable in time and space. For the total study interval, 11-11-77 to 3-22-84, twenty of the

Table 1. Goodwin Creek Scour (-) and Fill (+) Volume Changes (m^3)*

Reach Cross Section	Reach Length (m)	11-11-77 to 11-18-82	11-18-82 to 02-23-83	02-23-83 to 05-02-83	05-02-83 to 06-13-83	06-13-83 to 10-05-83	10-05-83 to 12-07-83	12-07-83 to 03-22-84	11-18-82 to 03-22-84	11-11-77 to 03-22-84
C1-1A	44.2	-1353	60	-269	237	76	-217	165	52	-1301
C2-1	59.7	-1451	-414	29	-326	203	-140	-224	-872	-2323
C2-5	74.7	-466	-54	160	144	-260	-122	-150	-282	-748
C3-2	136.9	-4913	-529	-227	655	-97	-493	-223	-914	-5827
C4-8	139.0	1340	98	368	333	392	-436	89	844	2184
C4-17	84.4	889	-258	-123	215	166	-238	-124	-362	527
C5-2	70.0	1401	81	102	-11	382	-100	21	475	1876
C5-6	60.5	317	-225	89	253	52	-49	-748	-628	-311
C5-10	75.3	-204	36	945	-659	-489	14	244	91	-113
C6-6	89.5	236	58	-88	-124	-188	-129	475	4	240
C7-4	110.2	-16	86	195	-636	-476	706	272	147	131
C8-5	135.9	630	81	-15	-304	314	-220	230	86	716
C10-1	145.5	77	354	832	-321	584	140	-83	1506	1583
C41-3	147.4	-1221	242	-446	-416	1005	-1000	92	-523	-1744
C42-3	142.0	-4055	-851	405	-92	86	-207	-1261	-1920	-5975
C43-2	129.1	-12125	-802	453	2088	789	-881	-2842	-1195	-13320
C45-1	120.1	-9007	-411	450	517	1817	-1351	351	1373	-7634
C46-1	121.2	-381	118	514	-244	1299	-1397	47	337	-44
C47-2	134.1	-408	298	116	-711	569	198	-163	307	-101
C50-1	163.4	-244	-183	247	-3370	364	-2013	1440	-3515	-3759
C52-1	178.2	-2146	16	143	-1147	1094	-561	-2	-457	-2603
T1-1	169.2	-1974	-106	-101	-2038	833	-167	212	-1367	-3341
T2-2	162.3	-1590	-352	548	-631	277	-298	360	-96	-1686
T3-5	195.7	- 941	434	-370	185	2481	-190	-395	2145	1204
T5-T5A	198.1	-6015	-1256	-402	1471	-698	695	-2086	-2276	-8291
T7A-T5	179.5	-2428	1495	-1542	889	1011	-2681	-2051	-2879	-5307
T7A-RG	205.6	-2271	-878	253	-1742	29	-942	621	-2659	-4930
T9	187.1	2	591	-415	689	-1335	844	-161	213	215
T11-1	138.1	456	-40	-43	-121	-1186	1303	-85	-172	284
T12-3	62.6	-276	81	26	229	-61	-116	66	225	-51
TOTALS	3859.5	-48137	-2230	1834	-4988	9033	-10048	-5913	-12312	-60449

* Values are given as computed, however, they are only significant to the nearest 10 m^3. Data for the intervals from 11-11-77 to 6-13-83 have been previously published (Murphey and Grissinger, 1985). Several of these values are revised.

thirty cross sections had a net loss of bed and bank material. The net loss of material for the 3.86 km length of channel was about 6×10^4 m^3, or about 15.7 m^3 per meter of channel length (m^3/m) for the 6.36 year study interval. For short-term intervals, channel fill (positive volume change) values are more frequent and, for two of the intervals, the channel experienced net fill. As an example of this short-term variability, reach C43-2 vascillated from a 2090 m^3 fill for one 42 day interval to a 2840 m^3 scour for a separate 106 day interval. Patterns of change, however, are not readily discernable.

In an attempt to explain at least some of this variation, the volume changes for the individual channel cross sections were organized (Table 2) into hydraulically similar reaches (Fig. 3) based on average velocity versus peak discharge relations. This functional classification is basically a generalization of the visual classification of zones A through M except for the added special relation of entrances to contracted reaches (Murphey and Grissinger, 1983). For Table 2, the five most-downstream cross sections (C1-1A to C4-8) and the four most-upstream cross sections (T7A-RG to T12-3) were deleted due to possible confounding influences of the gaging station flumes or to backwater from a highway bridge upstream of gaging station 2 or from Long Creek. The deletion of these nine cross sections and accompanying 1.05 km channel length resulted in a marginal increase in computed net annual scour, from 2.46 m^3/m to 2.68 m^3/m, but did not otherwise affect the general features of the data.

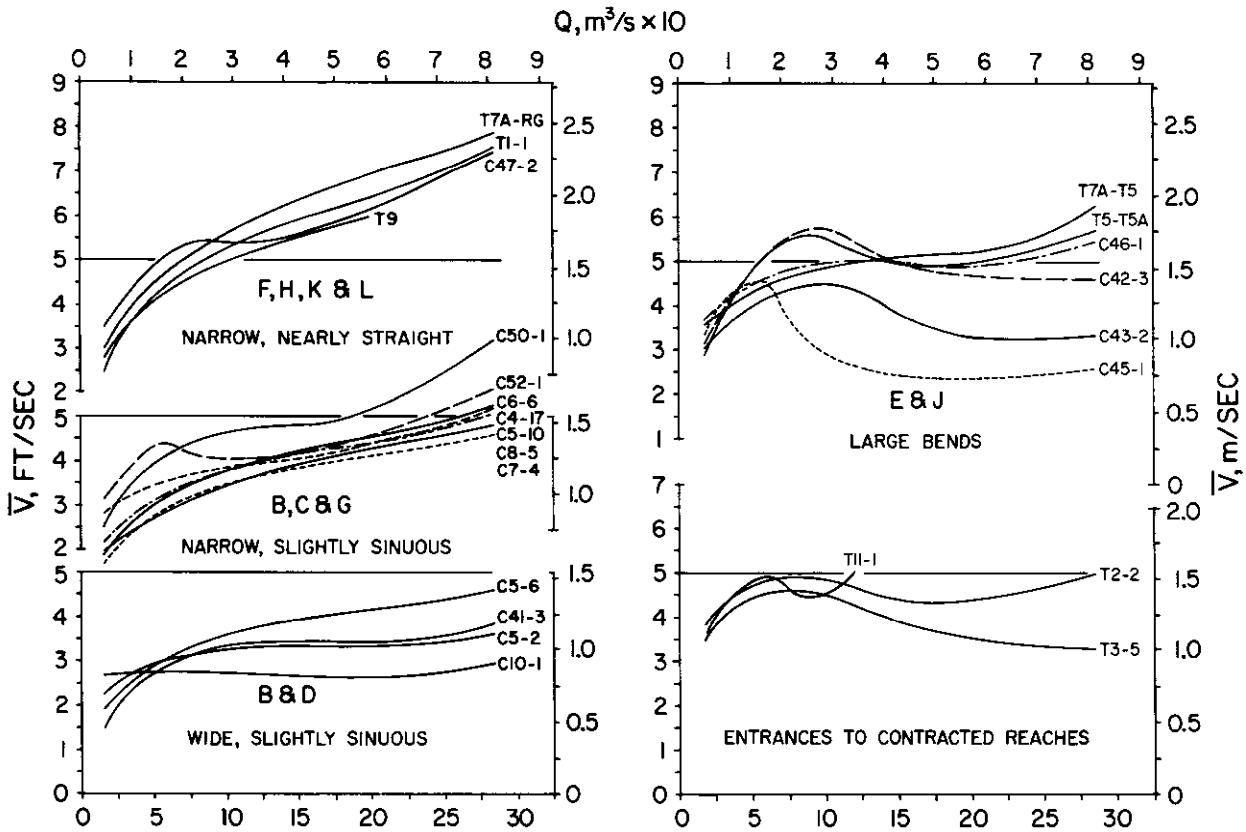

Fig. 3. Mean Velocity vs Peak Discharge for hydraulically similar reaches after Murphey and Grissinger, 1983.

Table 2. Summary of volume changes for hydraulically similar reaches as identified in Fig. 3*.

Survey Interval	Narrow Nearly Straight	Narrow Slightly Sinuous	Wide Slightly Sinuous	Large Bends	Entrances to Contracted Reaches	Totals for 21* Reaches
Fill - Scour = Net Volume Change						
	-------------	-------------	-------------	m^3	-------------	-------------
Interval 1, 11/77-11/82** (1833 days)						
	0	1,760	1,800	0	0	3,560
	-2,380	-2,610	-1,220	-34,010	-2,530	-42,750
	-2,380	-850	580	-34,010	-2,530	-39,190
Interval 2, 11/82 - 2/83 (97 days)						
	300	280	680	1,610	430	3,300
	-110	-440	-230	-3,320	-350	-4,450
	190	-160	450	-1,710	80	-1,150
Interval 3, 2/83 - 5/83 (68 days)						
	120	1,530	1,020	1,820	550	5,040
	-100	-230	-450	-1,940	-370	-3,090
	20	1,300	570	-120	180	1,950
Interval 4, 5/83 - 6/83 (42 days)						
	0	220	250	4,970	190	5,630
	-2,750	-6,240	-750	-340	-630	-10,710
	-2,750	-6,020	-500	4,630	-440	-5,080
Interval 5, 6/83 - 10/83 (114 days)						
	1,400	1,940	2,020	5,000	2,760	13,120
	0	-1,150	0	-700	0	-1,850
	1,400	790	2,020	4,300	2,760	11,270
Interval 6, 10/83 - 12/83 (63 days)						
	200	720	140	700	0	1,760
	-170	-3,160	-1,150	-6,520	-490	-11,490
	30	-2,440	-1,010	-5,820	-490	-9,730
Interval 7, 12/83 - 3/84 (106 days)						
	210	2,660	110	400	360	3,740
	-160	-130	-830	-8,240	-400	-9,760
	50	2,530	-720	-7,840	-40	-6,020
SUBTOTAL, 11/82 - 3/84 (490 days)						
	2,230	7,350	4,220	14,500	4,290	32,590
	-3,290	-11,350	-3,410	-21,060	-2,240	-41,350
	-1,060	-4,000	810	-6,560	2,050	-8,760
TOTAL, 11/77 - 3/84 (2323 days)						
	2,230	9,110	6,020	14,500	4,290	36,150
	-5,670	-13,960	-4,630	-55,070	-4,770	-84,100
	-3,440	-4,850	1,390	-40,570	-480	-47,950

* The fill and scour values are sums of individual reaches having net fill or scour respectively. Reaches C4-8 and downstream and T7A-RG and upstream have been deleted due to local influences of the flumes on reach adjustments.

** No gaging data available.

Frequencies of net volume changes for the 21 reaches summarized in Table 2 are presented in Fig. 4, arranged in order of increasing survey interval. In general, the frequency of fill decreased and the frequency of scour increased for the survey intervals greater than 114 days. More significantly, the scour volume changes increased in magnitude with increased survey interval, indicating that scour occurred selectively at certain reaches. As summarized in Table 2, the net volume changes for the total survey interval (11/77 to 3/84) also show this selectivity. About 85% of the total net volume change (scour) occurred at large bends, primarily from bank scour of late-Holocene material. These locations represent about 32% of the total channel length, with all reaches in zones E and J (Fig. 2). For the 490 day gaged interval (Table 2), scour from the large bend reaches was actually more than double the total net scour from this 2.81 km length of Goodwin Creek. These results illustrate the utility of the classification of channel reaches by zones based on their observed forms.

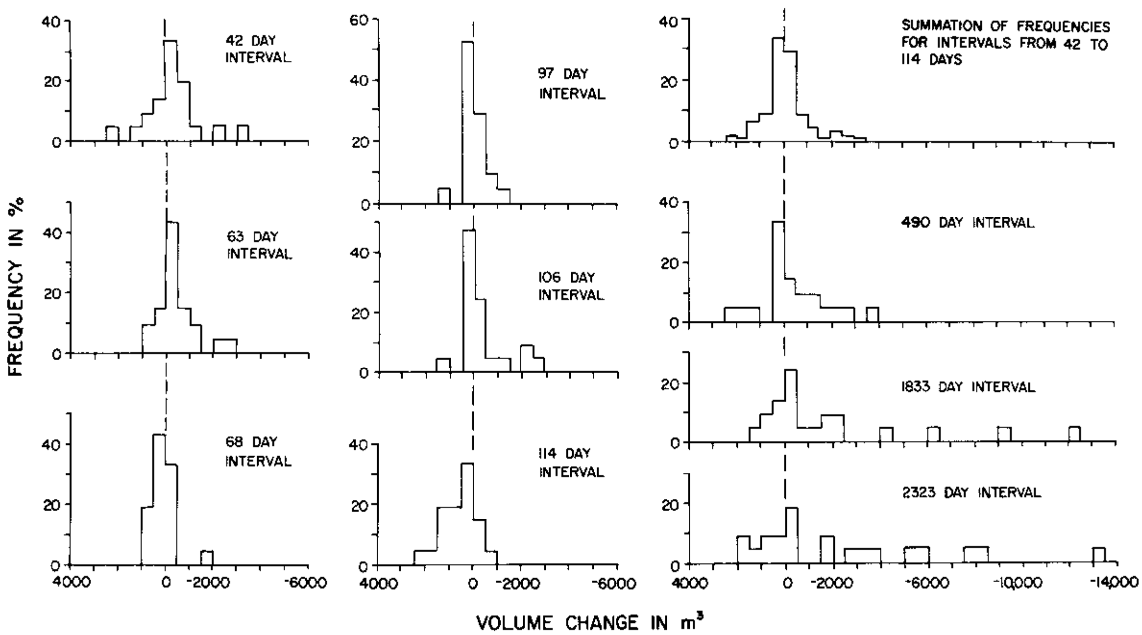

Fig. 4. Frequencies of volume changes [scour (-) or fill (+)] during different lengths of survey intervals.

Differences in net volume change totals for the six short-term survey intervals (Table 2) and comparable frequency variations for the fill and scour volumes (Fig. 4) are not as easily explained. The restricted frequency distribution for the 114 day interval (6/83 to 10/83) and the net total fill (+11,300 m^3) for this interval are obviously due to the low runoff magnitude (Table 3). Nearly two thirds of the runoff for this interval resulted from only one storm event. The 97 and 68 day intervals (11/82 to 2/83 and 2/83 to 5/83, respectively) have comparable restricted frequency distributions (Fig. 4) and together produced a net fill of 800 m^3 for this 490 day interval. These two intervals, however, comprise 34% of the 490 day study interval and 54% of the total runoff for this interval (Table 3). In contrast, the greatest net scour for this 490 day interval occurred during the 63 day interval (10/83 to 12/83), encompassing 13% of the total gaged interval and 14% of the total runoff. For this 490 day study interval, the net volume changes for the individual short-term intervals were not significantly related to peak discharge or to runoff volumes listed in Table 3.

Table 3. Hydrologic data for each survey interval. Precipitation values are averages for 37 gages in & surrounding Goodwin Creek watershed. Runoff data is from Flume 1.

Survey Interval	No. of Days	Precip. * (mm)	Runoff Totals (mm)	Runoff Percent (%)	Runoff >2.54 mm/d (mm)	Runoff >6.35 mm/d (mm)	Runoff >12.7 mm/d (mm)	Runoff >25.4 mm/d (mm)	Peak Discharge (m^3/s)
11/77-11/82	1833	6980	---Data Incomplete---						
11/82-02/83	97	695	497	71.5	454	428	386	318	78.6
02/83-05/83	68	326	206	63.2	171	153	123	93	68.2
05/83-06/83	42	333	186	55.9	172	168	161	147	71.3
06/83-10/83	114	306	75	24.5	50	50	50	50	107.0
10/83-12/83	63	412	184	44.7	171	160	125	125	96.3
12/83-03/84	106	340	165	48.5	119	90	56	0	19.2
11/82-03/84	490	2412	1313	54.4	1137	1049	901	733	107.0

* The 98-year average annual precipitation at Batesville 2SW gage was 1360 mm. Precipitation for the 6.36 year period averaged 1475 mm/year or 115 mm/year (8%) above average. The average annual precipitation for the 490 day interval (11/82 - 3/84) was 1797 mm/year or 437 mm/year (32%) above average; that for the 11/77 to 11/82 interval was 1390 mm/year or 30 mm/year (2%) above average.

Gross volume changes, both fill and scour, for the individual short-term intervals are presented in Fig. 5 in relation to total runoff volume. Log-log plots were used to evaluate the possibility of regime type relations of the general form

$$VC \sim aR^b \tag{1}$$

where VC is the volumetric change and R the total runoff volume. Neither volume change is significantly related to total runoff volume (Fig. 5) or to any of the other runoff quantities listed in Table 3. Rather, we believe these results illustrate the influence of variations of sediment source availability. The two intervals having relatively low gross fill volumes (intervals 6 and 7, Fig. 5) immediately followed the interval of maximum gross and net fill (interval 5, Fig. 5). Presumedly, sediment availability from the contributing channel system had been temporarily depleted, limiting coarse sediment input to this study channel. As would logically follow, the relatively large volumetric scour values for intervals 6 and 7 (Fig. 5) would be explained by the combined influences of maximum availability of sediment temporarily stored in the study channel and the reduced input of coarse sediment from the contributing channel system. Similarly, although less clearly defined due to limited preceding data, the relatively large scour volume for interval 4 (Fig. 5) immediately followed a period of 800 m^3 net fill (intervals 2 and 3, Table 2).

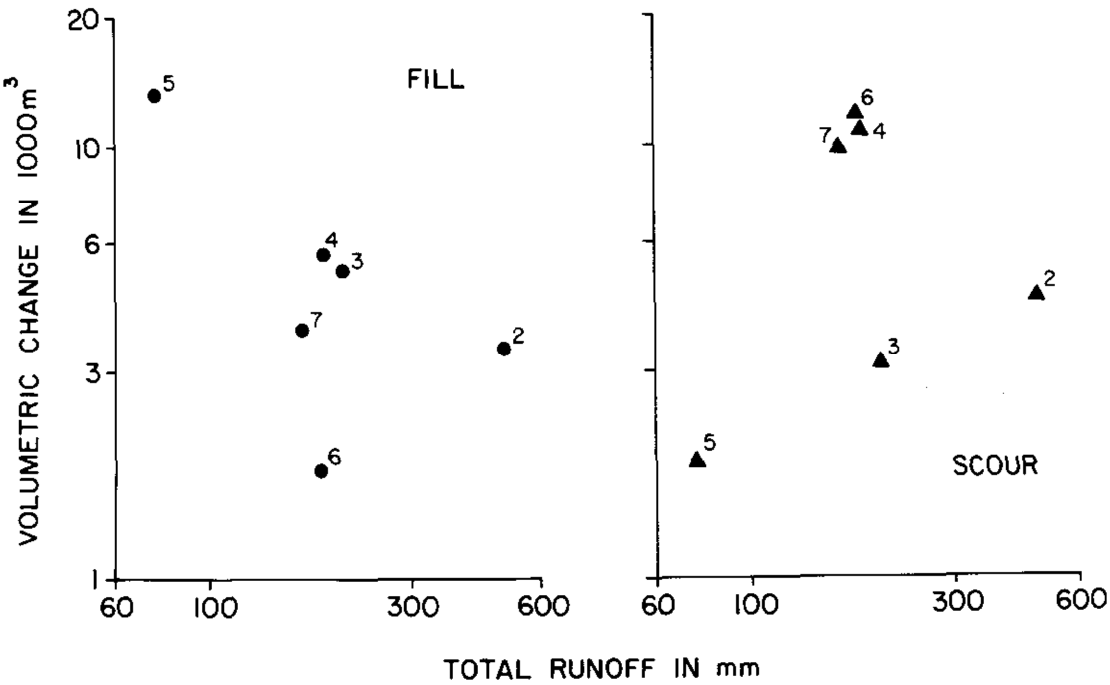

Fig. 5. Relationships of scour and fill to total runoff.

The concentration of bank scour in the large bendways of zones E and J and the prevalence of late-Holocene material at these sites, in contrast to more stable early-Holocene materials at other sites, indicates that bank material lithology is an additional control. Late-Holocene materials are subject to tension crack accentuated mass failure. The critical conditions for this type of failure at constant bed elevation are (1) tension crack development, (2) minimum strength worst case saturation, and (3) bank oversteepening. Whereas the first two conditions vary with cumulative climatic controls, bank oversteepening is controlled by hydraulic conditions of channel flow events but not necessarily on an individual storm basis. Certainly, the rate of bank toe removal leading to bank oversteepening will vary with the concurrent rate of sediment storage (fill) or scour but this influence could not be evaluated at this time.

SUMMARY AND CONCLUSIONS

Channel adjustments during this 6.36 year study period have been a significant source of sediment, affecting both the study channels and the sediment supply to downstream channels. For this total interval, the net annual scour rate for the 2.81 km channel length between discharge measuring flumes 1 and 2 (Fig. 1) was 2.68 m^3/m of channel or 7,500 m^3/year. This rate is equivalent to about 5.5 metric tons per hectare (2.4 tons/acre) annual erosion for the contributing watershed area. For the initial 5.02 year ungaged interval, the comparable rate was 2.78 m^3/m and for the 1.34 year gaged interval the rate was 2.32 m^3/m, even though precipitation for this latter interval was 29% greater than that of the initial 11-77 to 11-82 interval. Short-term gross and net volume changes are much more variable presumedly due to variations in the supply of coarse sediment input to the study channel and to variations in the amount of coarse sediment previously stored in the study channel.

The short-term net and gross volume changes were not significantly related to runoff volumes. Net scour, primarily bank scour, was concentrated at select locations identified by both channel form and the average velocity versus discharge relations. About 85% of the total net scour occurred at large bend locations (zones E and J) constituting about 32% of the total channel length. The bank materials in these reaches are primarily late-Holocene materials subject to tension crack accentuated mass failure. These results indicate lithologic control of channel form and bank adjustments for this incised channel. With respect to bank instability, the effects of individual runoff events on bank failure are largely restricted to slough removal leading to bank oversteepening. The development of worst case saturation conditions and/or tension cracks are the primary controls and are not related to individual runoff events. Both changes (fill and scour) must be addressed as separate aspects in any study of channel adjustment. Obviously, this will necessitate consideration of hydraulic and non-hydraulic variables additional to those discussed in this report.

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SONAR PATTERNS OF COLORADO RIVER BED, GRAND CANYON

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ABSTRACT

Distinctive patterns on side-scan sonar charts and depth-finder charts are used to delineate smooth bottom, sediment waves, and boulders and bedrock outcrops on the bed of the Colorado River in the Grand Canyon, Arizona. Dredged bed-material samples indicate that sand and gravel form smooth bottom in areas of low-velocity currents and sediment waves in areas of intermediate velocity. Gravel and cobbles form smooth bottom in areas of higher velocity near riffles, rapids, or downstream sides of scour holes. Near the banks, smooth bottom is mainly sand and commonly extends above the water surface as sand banks. Sediment waves range in amplitude from 0.2 to 1.2 meters. The bed-material samples indicate that sediment waves are composed mainly of medium to very coarse sand, fine gravel, and a few medium to large pebbles. The granules and pebbles are mostly subrounded to well rounded and show evidence of sustained transport. Boulders and bedrock outcrops occur at riffles, rapids, upstream sides of scour holes, and along much of the banks. River depths range from 1.5 to 32.3 meters at an average discharge of 708 cubic meters per second. Scour holes more than 1.5 times deeper than local mean depth occur below many rapids and constrictions. The holes contain cobbles and gravel in the bottom and store little sand. In metamorphic-rock reaches, the river bed is irregular and has near-vertical changes in depth of as much as 15 meters.

INTRODUCTION

The sand banks and bars, locally called "beaches," along the Colorado River in the Grand Canyon, Arizona, are composed of sand that is both deposited and eroded by the river. The river bed serves as a reservoir of sand that is available to be transported to the beaches. The sand is one component of the movable sediment that is transported by the river. Large volumes of sediment enter the river and are transported during floods; sustained flow sorts and redistributes the material on the river bed. Although small boulders and cobbles move during floods, subrounded to well-rounded gravel and sand constitute the movable sediments at lower sustained discharges. In this study, distinctive patterns on side-scan sonar charts and depth-finder charts are used to delineate the distribution of movable sediment that forms parts of the river bed. Bed-material samples were collected to provide qualitative description of the sediment and calibration of bed material to side-scan sonar patterns. These data are part of the input to the sediment-transport model used by the U.S. Bureau of Reclamation and the U.S. Geological Survey to define the movement of sediment through the canyon. The sediment-transport model is discussed in J. B. Graf's paper "Sediment Studies in Grand Canyon, Arizona," this volume.

PREVIOUS WORK

The first detailed survey of the Colorado river in the Grand Canyon was done by the Geological Survey in 1923 using alidades and plane tables (Birdseye, 1923). Maps showing river miles below the Survey gage at Lees Ferry and profiles of the water surface adjusted to a discharge of 283 m^3/s were produced. Water-surface profiles were adjusted by discharge because the stage commonly changes more than 3 m in response to changes in discharge. The first systematic measurements of river depths were made in 1965 by Leopold (1969). Leopold measured depths about every 0.1 mile using a nonrecording depth finder at a discharge of about 1,360 m^3/s. Locations were from aerial photographs, and mileage was from the 1923 Geological Survey river maps. Leopold described scour holes and discussed their formation by high-velocity downward-directed flow below rapids. In 1975, Dolen and others (1978) made a continuous-depth profile with a recording Fathometer* using the same location methods as used by Leopold; location errors were as great as 0.06 mile. Discharge during their trip was about 450 m^3/s. Howard and Dolen (1981) discussed distribution and transport of sand, cobble, and boulders by current flow and the change in sediment transport caused by construction of Glen Canyon Dam. They related origin and location of scour holes to both geologic structure and position below rapids and constrictions.

DATA COLLECTION

Data for this study were collected during three trips on the Colorado River for 236 miles below Lees Ferry (Fig. 1). Side-scan sonar images and depth profiles were run concurrently March 1-10, 1984; discharge during that trip was 688 to 731 m^3/s and averaged 708 m^3/s. Cross sections were taken April 28-May 7, 1984, while discharge was 722 to 940 m^3/s. Bottom samples were collected September 4-11, 1984; discharge was 680 m^3/s. Locations along the river were determined by annotating about 1,050 navigation points on aerial photographs taken in 1973 and on recorder charts as data were collected. River miles were assigned to the navigation points on the basis of the 1923 Geological Survey river maps (Birdseye, 1923). Location accuracy is within 0.03 mile.

A Klein 100-khz side-scan sonar unit was used to acquire acoustic images of about 80 percent of the river bed. The side-scan sonar fish was removed from the water for many of the larger rapids. Images made in or just below rapids or large riffles were unusable probably because air bubbles entrained in the water absorbed the returning acoustic signal. A Raytheon 208-khz Fathometer was used to record a depth profile of about 95 percent of the river. The profile was taken concurrently with the side-scan sonar data. An attempt was made to measure the depth profile near the thalweg. An assumed sound velocity in water of 1,460 m/s was used to convert travel time to depth or to distance. Cross sections of the river were taken in pools between rapids or large riffles at 224

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*Use of brand names in this report are for identification purposes only and does not constitute endorsement by the U.S. Geological Survey.

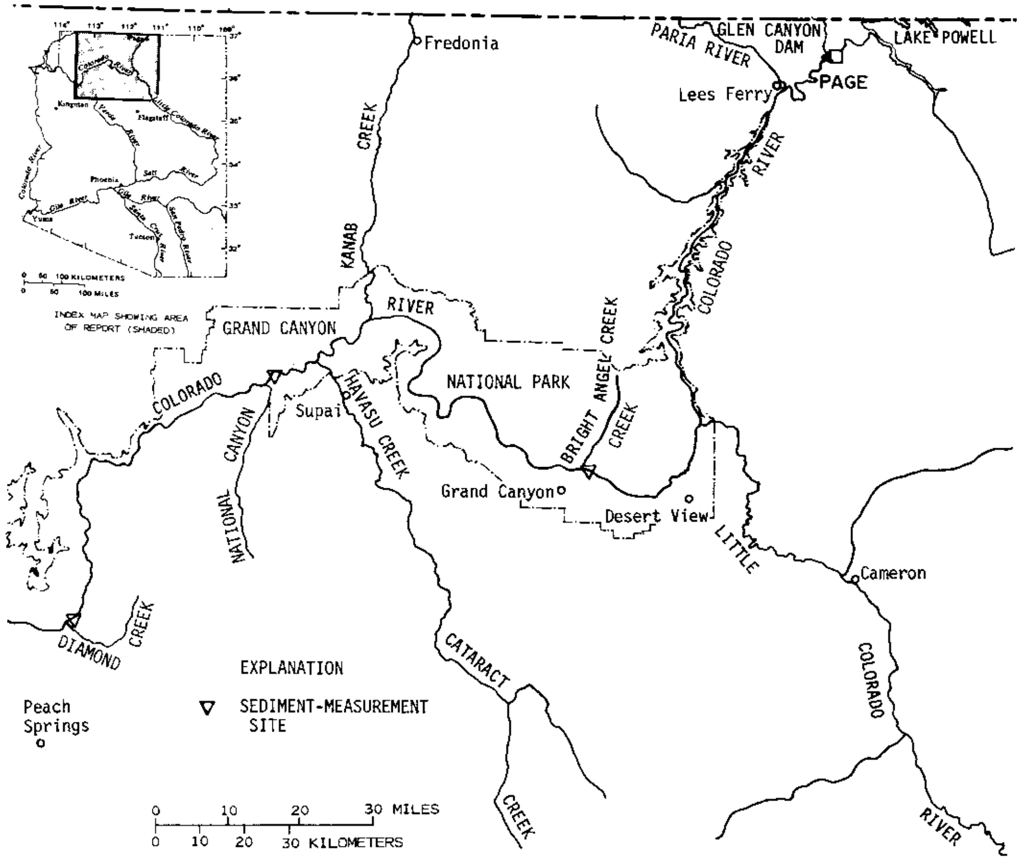


Figure 1. Study area.

locations with the Fathometer and a continuous seismic (subbottom) profiler. The sonic pulse of the profiler was produced by a boomer and had most of its energy concentrated in the frequency range of 900 to 2,000 hz.

Fifty bed-material samples were taken with a 10-inch pipe (255-mm inside diameter) dredge in order to calibrate patterns delineated on the side-scan sonar charts to the bed material. Sampling depths ranged from 3.6 to 19.5 m. Quantitative samples of sand and fine gravel and qualitative samples of large pebbles and cobbles were taken by the dredge. Two small boulders were recovered. A typical sample volume was 0.1 to 0.14 m^3. At several sites, bed material was not recovered in the dredge presumably because the river bed consisted of bedrock outcrops or boulders too large to enter the dredge. Positive contact with the river bed was made during three sampling attempts before a "no return" was declared. Bed-material samples taken with BM-54 samplers at five sediment-collection stations (Graf, this volume) also were used in calibration of the sonar patterns.

SIDE-SCAN SONAR PATTERNS

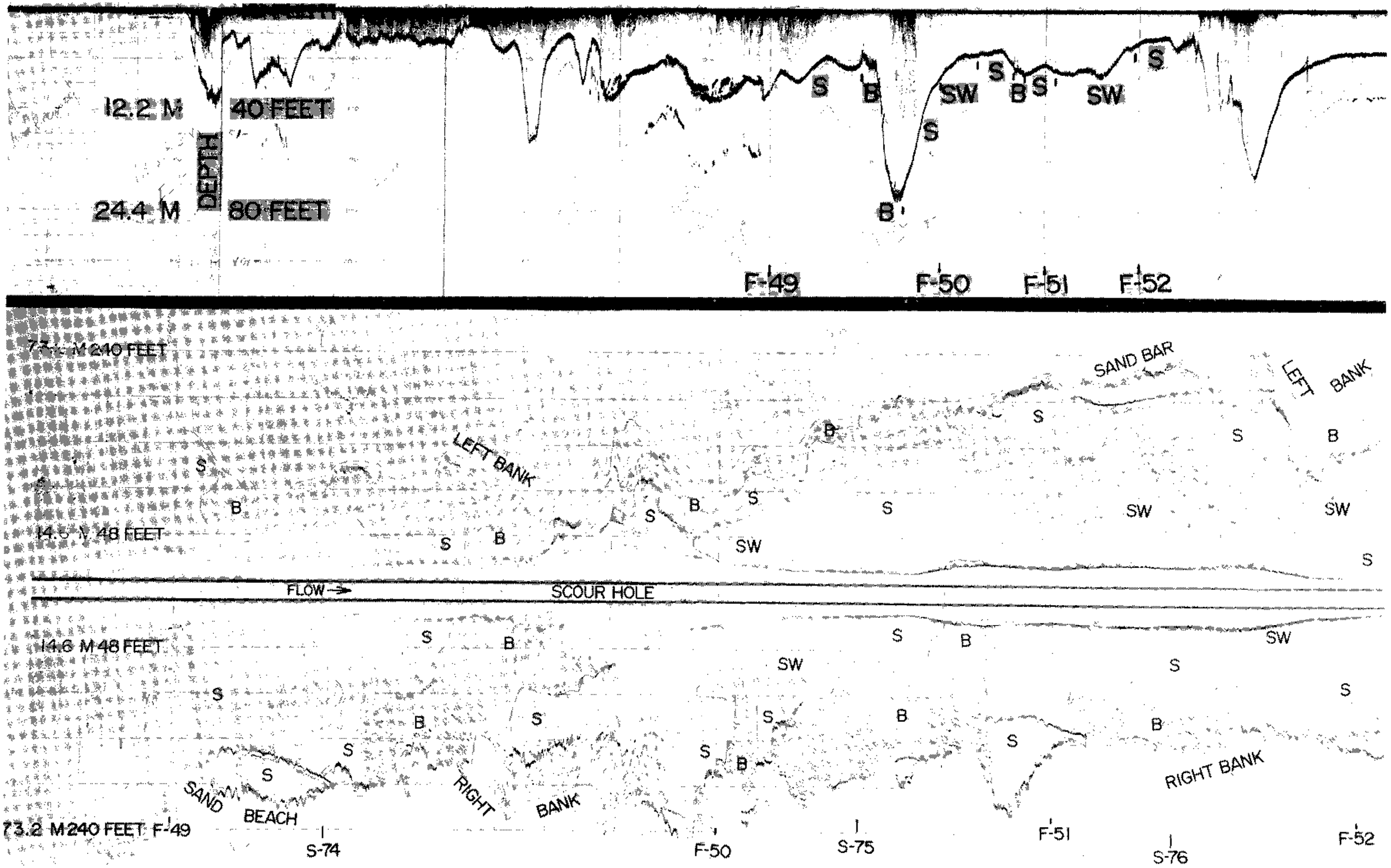
A side-scan sonar pattern is produced by multiple acoustic reflections from particles and surface features of an area of river bed. The location and shape of accumulations of movable sediment on the river bed that produce side-scan sonar patterns are a function of sediment particle size, current-velocity pattern, and length of time a particular velocity pattern has existed. The particles are sorted and distributed in an organized way in the sediment accumulations as a result of the variations in local current velocity. Side-scan sonar images on the chart are divided visually into three patterns—B, boulders and bedrock; S, smooth bottom; and SW, sediment waves (Fig. 2).

Boulders and bedrock outcrops are large in size compared to the resolution of the sonar unit and produce a bold broken pattern of stripes and spots (pattern B). A diver's observations at a test section above Lees Ferry; BM-54 bed-material samples at the Grand Canyon, National Canyon, and Diamond Creek sediment-collection stations (Fig. 1); and observations at cross sections indicate that the boulders and bedrock outcrops are commonly covered with a thin layer of sand. The irregular shapes, however, show through the sand to produce the B-pattern. Boulders and bedrock are most extensive along the banks where talus enters the water or bedrock outcrops are exposed. The river bed of rapids and of the upstream side of scour holes is commonly composed of boulders or bedrock. Boulders form the bed of some riffles. Bedrock outcrops that project above the bottom sediments or large boulders that rest on the river bed create isolated occurrences of pattern B.

Smooth bottom (pattern S) consists of sand, gravel, or cobbles that are smaller than the smallest particle that can be resolved by the sonar unit. The pattern is typically pale, low contrast, and stippled. Pattern S generally is shaded because the return signal gradually increases in strength as the distance from the fish (range) increases. The ripples and irregularities of the river bed return a stronger signal as the angle of incidence increases. Areas of the bed near exposed sand beaches and bars commonly have underwater cutbanks that show on the sonar charts as bold smooth stripes. These areas are included in the smooth-bottom pattern.

Low-flow aerial photographs taken in October 1984 and dredged bed-material samples show that cobble and gravel cannot be distinguished reliably from sand on the side-scan sonar charts. The inability to distinguish between the two bed materials probably is due to the limited resolution and the low dynamic response of the chart paper. In some areas, however, the image does appear to show two types of smooth bottom. An imaging system that would respond to the full dynamic range of the returning signal could probably distinguish sand bottom from gravel and cobble bottom.

Smooth bottom is found near the banks, on the downstream side of scour holes, in some riffles, and in shallow areas just above many rapids. The location of smooth bottom appears to correlate with low-current velocity near banks and higher velocity between areas of boulders and bedrock and of sediment waves. The sequence of patterns—boulders and bedrock to smooth bottom to sediment waves to smooth bottom to boulders and bedrock—is repeated many times through the canyon. Smooth bottom that extends from low-velocity areas to the beginning of sediment waves probably represents the tranquil-flow transition from plane bed to dunes



EXPLANATION

B SIDE-SCAN SONAR PATTERNS—B, boulders and bedrock; S, smooth bottom; SW, sediment waves

F-50 NAVIGATION POINT

Fig. 2. Uncorrected side-scan sonar and Fathometer charts.

(Simmons and others, 1961, p. 36). Smooth bottom near the banks is mainly sand and commonly extends above water where sand bars and beaches are exposed. The aerial photographs taken in October 1984 indicate that smooth bottom in the lower canyon is commonly composed of cobbles. Basalt flows provide an abundant supply of tough cobble- and boulder-sized clasts to the river below mile 179.

Sediment waves (pattern SW) are efficient reflectors of sound and produce a strong returning signal. The cyclic alternation of sloping surfaces produces a moderate- to high-contrast striped pattern. The sonar unit resolved sediment waves of amplitudes as small as 0.2 m; the largest amplitude observed was 1.2 m. Sediment waves are most common near the center of the river in areas of intermediate current velocity and depth. The waves typically begin downstream from short reaches of rising smooth bottom on the downstream side of scour holes and change back into smooth bottom at constrictions or riffles where the depth decreases and current velocity increases. Dredged bed-material samples indicate that the sediment waves are composed of medium to very coarse sand, fine gravel, and a few medium to large pebbles. The granules and pebbles show evidence of sustained transport; they are mostly subrounded to well rounded, smooth surfaced, and free from algae or coatings. The fine gravel probably is being transported as bed load; neither the samples recovered from sediment waves nor the behavior of the dredge during sampling indicated the existence of an armoured bottom.

DEPTH PROFILE

The depth profile of the river was taken concurrently with the side-scan sonar images and was used in interpretating the sonar patterns. The Fathometer chart showed sediment waves of amplitude of about 0.1 m and bottom features of about 0.2 m in size. The unit commonly showed depths and bed forms in turbulent areas where the side-scan sonar unit produced poor or no records. Profile depths ranged from 1.5 to 32.3 m at an average discharge of 708 m^3/s; mean depth was 11 m. The bottom is irregular in metamorphic-rock reaches with near-vertical depth changes of as much as 15 m.

Scour holes more than 1.5 times deeper than the local mean depth occur below many rapids and riffles. The scour holes are a local feature that owe their location to the high-energy level generated by fall through rapids. Scour is displaced and concentrated below the rapids because the boulder bed of the rapids is resistant (Howard and Dolen, 1981, p. 291). Many of the holes display a characteristic pattern on the depth profile. Rough bottom of boulders and bedrock slopes downward to the scour-hole bottom on the upstream side. Smooth bottom begins in the bottom of the hole or at the base of the downstream side and continues downstream as depths decrease until sediment waves appear. The slope of the downstream side changes smoothly from about one-half the upstream value to 0 (Fig. 3). Bed-material samples suggest that the bottoms of the holes consist of cobbles and gravel that become finer downstream and change to fine gravel and sand that form sediment waves. Only a small smount of sand is stored in the holes during a sustained discharge of 708 m^3/s.

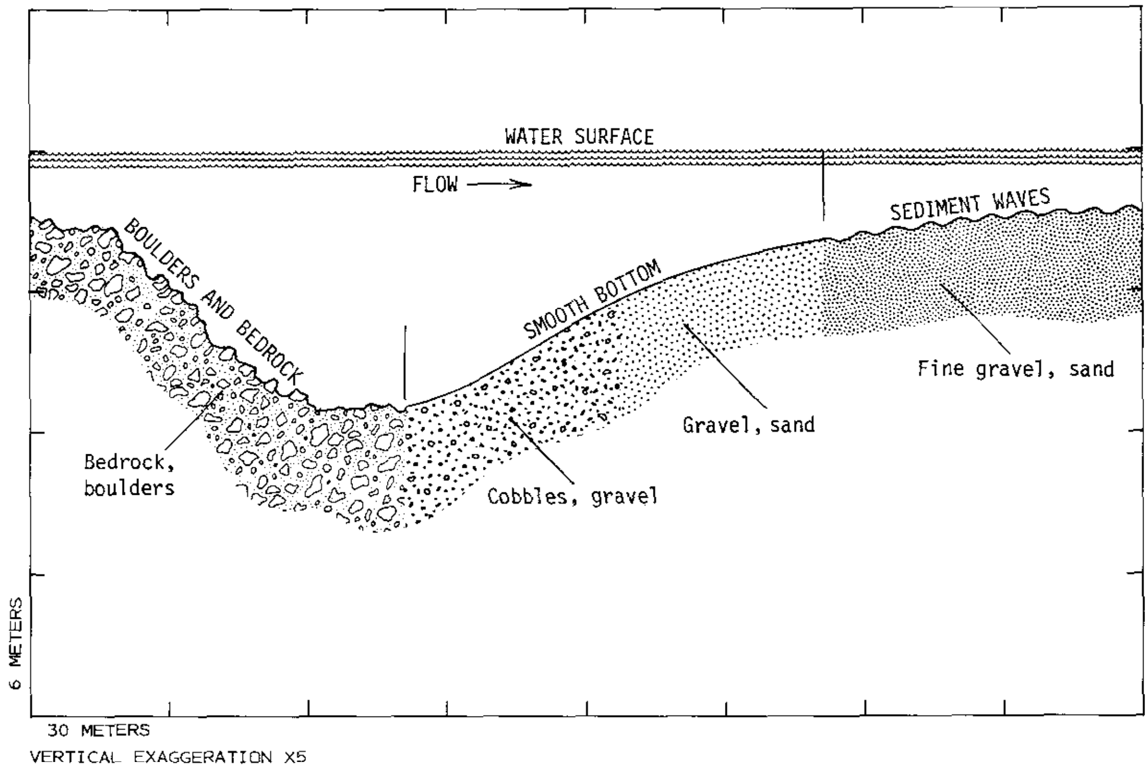
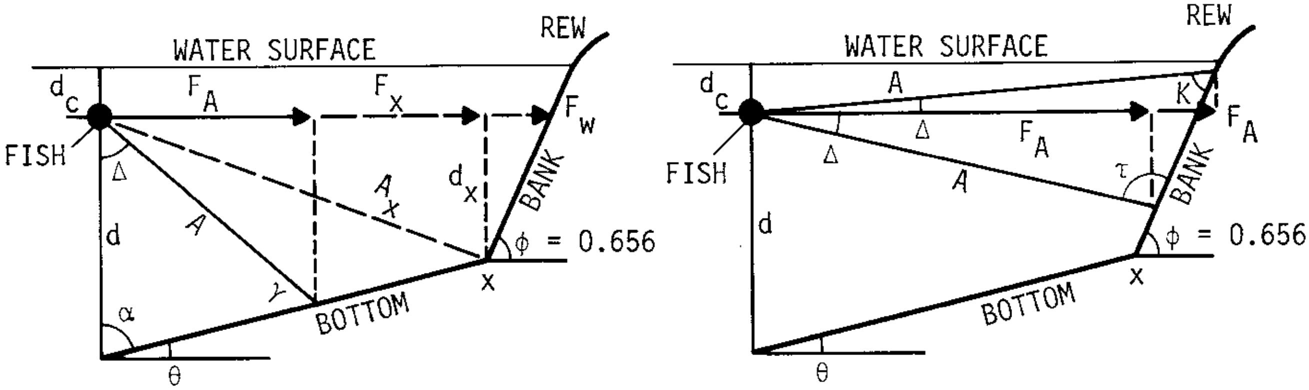


Figure 3. Generalized scour-hole profile, location of side-scan sonar patterns, and bed material.

Current velocity in the holes remains greater than in the pools downstream and prevents significant accumulation of sand. The sediment deposited in the hole by a previous flood probably coarsens downward to the point of maximum scour because the particles that accumulate on the bottom become smaller as velocity and discharge decrease during the flood recession.

ANALYSIS OF PATTERN DATA

The sonar patterns contain three distortions that must be corrected in order to make geometrically correct maps. The distance axis is traced out by the fish as the boat travels down the river. The plotting position of the distance-coordinate values (miles below Lees Ferry) vary because of changes in boat speed and flow velocity. The digitized distance coordinates were corrected by linear interpolation between navigation points using assigned mileages interpreted from the 1923 Geological Survey maps. Bottom reflections on each side of the fish plot on the range axis of the chart at a point farther than their true position from the distance axis. The range-coordinate value is decreased by an amount that varies from 0 near the river banks to the distance of the fish above the bottom for reflections directly beneath the fish. The corrections are determined from the depth profile and the bottom and bank slopes. Data from the 224 cross sections are used to estimate the slopes. Generalized cross-section geometry and range-correction equations are shown in Fig. 4. Slant-range variables are defined in Table 1. The third



$$F_w = \text{LEW or REW} - 1.30\, d_c$$

$$F_x = \frac{d - .77\, F_w}{\tan\theta - .77}$$

$$d_x = d - F_x \tan\theta$$

$$A_x = [F_x{}^2 + d_x{}^2]^{1/2}$$

LEFT AND RIGHT BOTTOM:

$d \lessgtr A \lessgtr A_x$

$\alpha = 1.57 - \theta$

$\sin\gamma = \dfrac{d \sin\alpha}{A}$

$\Delta = \gamma - \theta$

$F_A = A \cos\Delta$

LEFT AND RIGHT BANK:

$A_x \lessgtr A \lessgtr F_w$

$\sin\tau = \dfrac{F_w \; .61}{A}$

$\Delta = 3.14 - \tau - 0.656$

$F_A = A \cos\Delta$

LEFT AND RIGHT EDGE:

$F_w \lessgtr A \lessgtr \text{LEW or REW}$

$\sin K = F_w \sin(3.14 - 0.656)$

$\Delta = 0.656 - K$

$F_A = A \cos\Delta$

Figure 4. Generalized cross-section geometry and range-correction equations.

Table 1. Slant-range correction variables

Name	Symbol	Units	Explanation
slant range	A	m	Distance toward right or left bank from side-scan sonar fish to a given point on the river bed. Perpendicular to distance axis.
depth	d	m	Vertical distance from fish to bottom.
fish depth	dc	m	Distance below water surface to fish.
tan ϕ		-	Slope of bank of generalized section. Mean value of bank slopes of 224 sections is 0.77.
tan θ		-	Slope of bottom of generalized section. Tan Θ is a function of depth and F_w.
	LEW, REW		Right or left edge of water.
	α	radians	Complement of bottom angle, alpha = 1.57 -θ.
	γ	radians	Angle between bottom and slant range.
	Δ	radians	Angle between slant range and horizontal.
	θ	radians	Angle between horizontal and bottom of generalized section.
	ϕ	radians	Angle between horizontal and bank of generalized section.
	τ	radians	Angle between slant range and bank of generalized section.
	k	radians	Angle between slant range and banks of generalized section.
	F_A	m	Horizontal distance from fish to a given point of river bottom. Perpendicular to distance axis.
	F_x	m	Horizontal distance from fish to intersection of bottom and bank of generalized section.
	d_x	m	Vertical distance from fish to intersection of bottom and bank of generalized section.
	A_x	m	Slant range to intersection of bottom and bank of generalized section.
	F_A	m	Horizontal distance from fish to right or left bank of generalized section.

correction required is to bend the distance axis to conform to the actual curving course followed by the boat as it traveled down the river. The river is generally narrow with respect to bend radii, and only a few miles of wide meandering channel are present in the canyon. The error in computation of pattern areas caused by this distortion is probably small compared to the errors of range and distance. This third correction was not made and the patterns are plotted and areas are computed using the distance axis as a straight line.

The side-scan sonar-pattern boundaries were digitized by the Bureau of Reclamation as optional-format digital line graphs (Allder and Elassal, 1983). Editing, creating a topologically correct data set, computing areas, and plotting maps were done with ARC-INFO software.

CONCLUSIONS

Calibrated side-scan sonar patterns can be used to delineate boulders and bedrock, smooth bottom, and sediment waves on the bed of the Colorado River in Grand Canyon, Arizona. The patterns interpreted in this study, however, cannot be used to reliably differentiate sand from gravel on smooth bottom or sediment waves. A 10-inch pipe dredge can be used to collect bed-material samples of cobbles, gravel, and sand from smooth bottom or sediment waves, but many more samples will be required to define the fraction of sand in movable sediments. At least a 20-inch pipe dredge will be required to quantitatively sample the coarse-gravel fraction of the movable sediment. Movable sediment during a discharge of 708 m^3/s consists of sand and rounded gravel and forms smooth bottom or sediment waves. Scour holes more than 1.5 times deeper than local mean depth contain cobbles and gravel in and near the bottom; little sand is stored in them during that discharge.

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CHANNEL ADJUSTMENTS AFTER PASSAGE OF A LAHAR

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ABSTRACT

Pine Creek was altered by a large lahar generated during the May 18, 1980, eruption of Mount St. Helens. The nature, magnitude, and persistence of subsequent channel adjustments have been quantified using data from annual surveys of 27 channel cross sections established along the lahar-affected length of the stream and from post-eruption aerial photographs taken periodically.

Post-lahar channel adjustments occurred rapidly in response to storm runoff. The channel widened by 3 to 63 m (10 to 305 percent) at measured cross sections during 1980-81, concomitant with incision in upstream parts of the basin and aggradation and braiding in downstream reaches. Widening during subsequent years was negligible. In upstream reaches, initial incision was followed by a small amount of aggradation during the 1982-84 water years; in downstream reaches, braiding diminished as the initial aggradation was followed by channel incision and enlargement.

The lahar-modified channel has adjusted in response to a combination of factors: 1) heavy rainfall during November 1980, which affected upper parts of the basin that usually are covered with snow, 2) movement of a large volume of sediment from upstream to downstream reaches during the first year following the eruption, and 3) higher than normal flood peaks occurring during water years 1982-84 and possibly during water year 1981.

INTRODUCTION

The May 18, 1980, eruption of Mount St. Helens generated massive lahars (debris flows or mudflows originating on the slopes of a volcano) that flowed down channels draining the flanks of the volcano (Janda and others, 1981).

The lahar that flowed the length of Pine Creek (fig. 1) altered the size, shape, pattern, and stability of the existing channel. Pre- and post-eruption channel position coincided over much of the stream length. Locally, however, the channel was displaced from its pre-eruption position, shortened, and straightened. Roughness elements of the channel bed and banks as well as the pool-and-riffle patterns that existed prior to the eruption were eroded or buried, thereby increasing the hydraulic smoothness of the channel (Lisle and others, 1983). Lahar deposits as thick as 2 m in and near the channel reduced channel capacity and provided an abundant source of readily erodible sediment (Janda and others, 1981). Incision during lahar recession created vertical, unstable channel banks.

The long-term effects of large debris flows on fluvial channel morphology and stability are not well known. Subsequent channel adjustments, erosion, sedimentation, and sediment transport constitute potential hazards and may create sediment-management problems that persist for years after passage of a lahar.

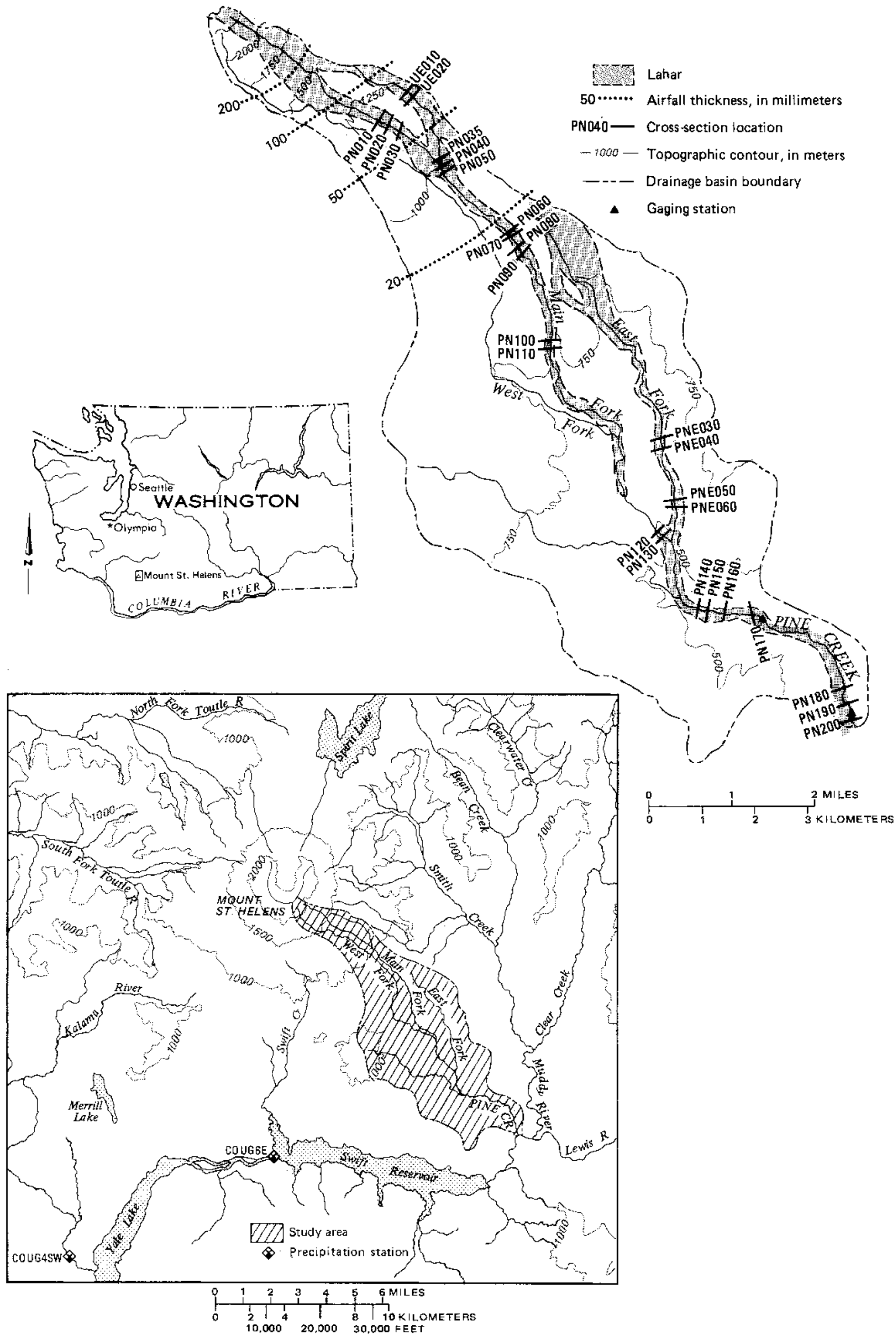

FIGURE 1. – Location of Pine Creek, Washington, showing extent of May 18, 1980 lahar, distribution of 1980 airfall deposits, and location of channel cross sections.

In this paper, analyses of post-eruption cross-section survey data and aerial photographs for Pine Creek are presented to identify processes dominating post-lahar channel adjustments through water year (WY) 1984. Pre- and post-eruption streamflow data are compared to evaluate the magnitude of post-eruption peak flows.

DESCRIPTION OF THE STUDY AREA

Pine Creek heads at an altitude of about 2,400 m above mean sea level on the Shoestring Glacier on the southeastern flank of Mount St. Helens and flows approximately 21 km to its confluence with the Lewis River at about 300 m altitude (figs. 1 and 2). Streamflow is perennial below about 800 m altitude. The main fork joins with subparallel western and eastern forks at altitudes of about 585 m and 485 m, respectively. The 61-km^2 basin is steep and elongated.

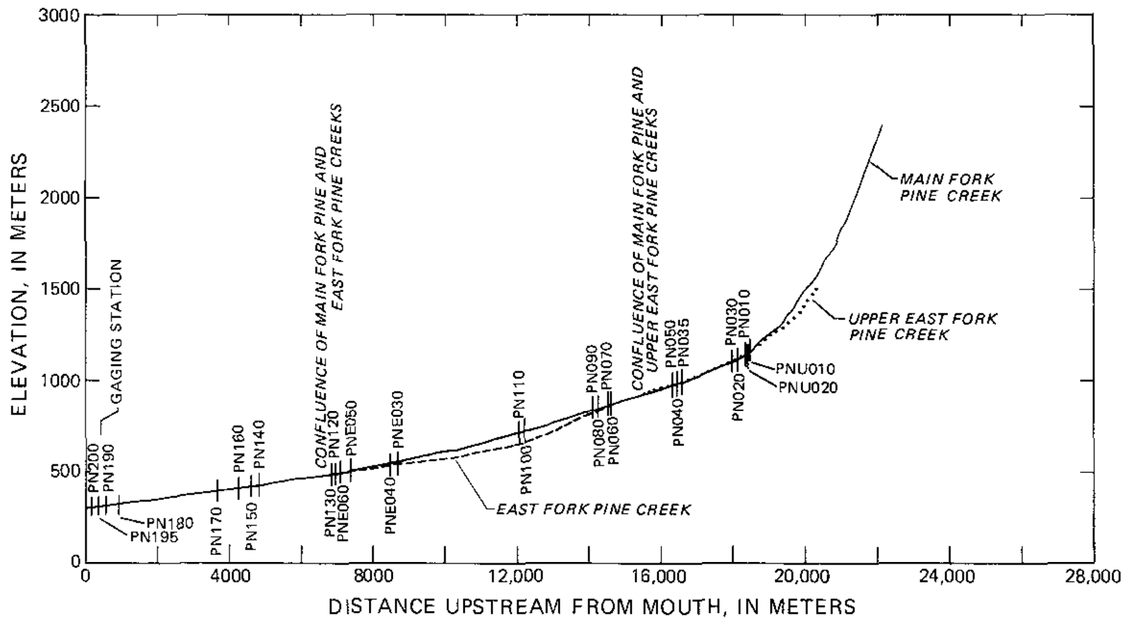

FIGURE 2. – Longitudinal profile of Pine Creek, showing locations of cross-section survey sites. Channel distance upstream from mouth and elevation above sea level are determined from U.S. Geological Survey topographic maps, 7.5-minute series, Mount St. Helens SE and Mount St. Helens NE quadrangles.

The two easternmost forks of Pine Creek (figs. 1 and 2) were affected by a large lahar that was generated within the first few minutes of the May 18 eruption and flowed the entire length of the channel. A general description of initiation, effects, and deposits of the lahar is given by Janda and others (1981). Initiation, flow behavior, and deposits of the lahar are described in detail by Pierson (1985).

The effects of the May 18 lateral blast and 1980 airfall deposits in the Pine Creek drainage basin are small compared to the effects of the lahar (fig. 1). Less than one percent of the drainage area was directly affected by the explosive phase of the May 18 eruption. Airfall deposits were thin and rapidly eroded or incorporated into the forest litter over most of the basin (Waitt and others, 1981; Swanson and others, 1983).

METHODS

Post-eruption changes in channel morphology were determined primarily by repetitive surveys of 27 cross sections established during the late summer and early fall of 1980 (figs. 1 and 2). Cross sections were located along the main and eastern forks of Pine Creek to sample the lahar-affected length of channel. Cross sections were resurveyed annually through 1984 during low flow. The methods used to establish the cross sections and analyze the data have been previously summarized (Martinson and others, 1985A and 1985B).

Channel location and channel pattern are documented by aerial photographs. Beginning in June 1980, channel-centered aerial photographs (1:9,600-scale) were taken for most of Pine Creek annually during low flow and after major winter storms. This post-eruption photography documents storm-induced planform changes of the channel and permits the extrapolation of changes measured at cross-section clusters to longer reaches.

A stream-gaging station was installed on Pine Creek during the summer of 1981 for determination of water and suspended-sediment discharge. This station is located about 3 km downstream from a former gaging station that was operated during WY 1958-72. In order to compare post- and pre-eruption instantaneous peak discharges at each site, applicable discharges were divided by the drainage area above the gaging station to compute discharge per square kilometer of drainage area (basin yield).

POST-ERUPTION STREAMFLOW AND HYDROLOGY

Annual instantaneous peak discharges at the post-eruption gaging station for WY 1982, 1983, and 1984 were 118, 90, and 85 m^3/s, respectively.

Comparison of pre- and post-eruption instantaneous peak flows adjusted for drainage area above the measurement site suggests that post-eruption annual peak flows are among the largest recorded for the drainage basin (fig. 3).

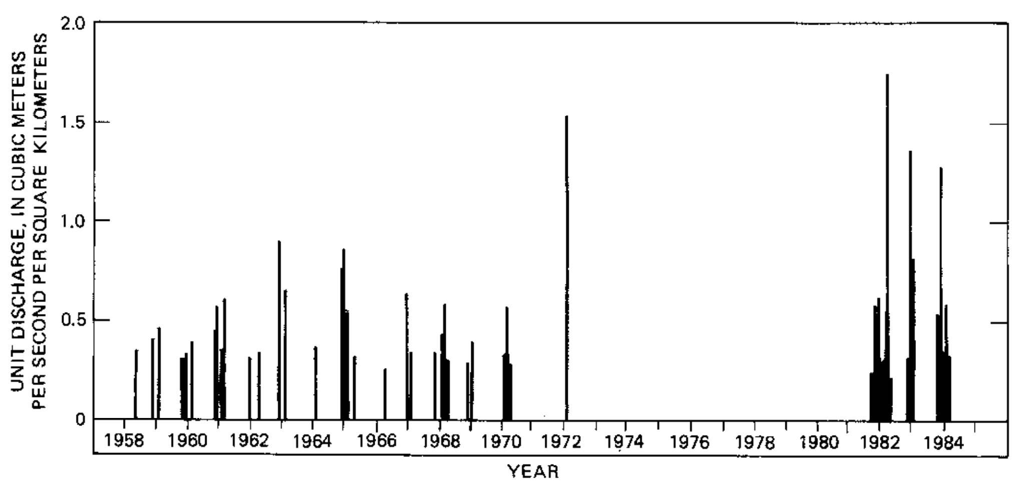

FIGURE 3. – Pre- and post-eruption instantaneous peak flows above base (17 m^3/s) adjusted for drainage area above measurement site, Pine Creek, Wash. The peak discharge value used for determining the WY yield was estimated 1972 basin using a water level and straight-line extension of the pre-eruption stage-discharge relation.

Pre-eruption peak flows had been estimated by extending a stage-discharge relation defined only for water discharges less than about 14 m^3/s. Therefore, the estimates of pre-eruption peak flows may not be accurate.

Regional analysis of the long-term 24-hr precipitation data show that no unusually large storms occurred during WY 1981-84. The two-year 24-hour total precipitation of about 127mm (Verne Bissel, National Weather Service, 1983, oral commun.) has not been exceeded during the period 1980-84 at Cougar 6E, the long-term weather-data site nearest the study area (fig. 1). The first major storm (November 6-7, 1980) after the eruption, however, appears to have been unusually large and intense for so early in the rainy season (Pacific Northwest River Basins Commission, 1969; National Oceanic and Atmospheric Administration, 1980-84). Precipitation during this storm occurred as rainfall on upper parts of the basin.

POST-ERUPTION CHANNEL ADJUSTMENTS

Adjustments of the lahar-affected channel to storm runoff occurred rapidly. Initial changes during WY 1981 included pervasive channel widening, incision in upstream reaches, and aggradation in downstream reaches (fig. 4). Subsequent channel adjustments during WY 1982-84 were dominated by aggradation and

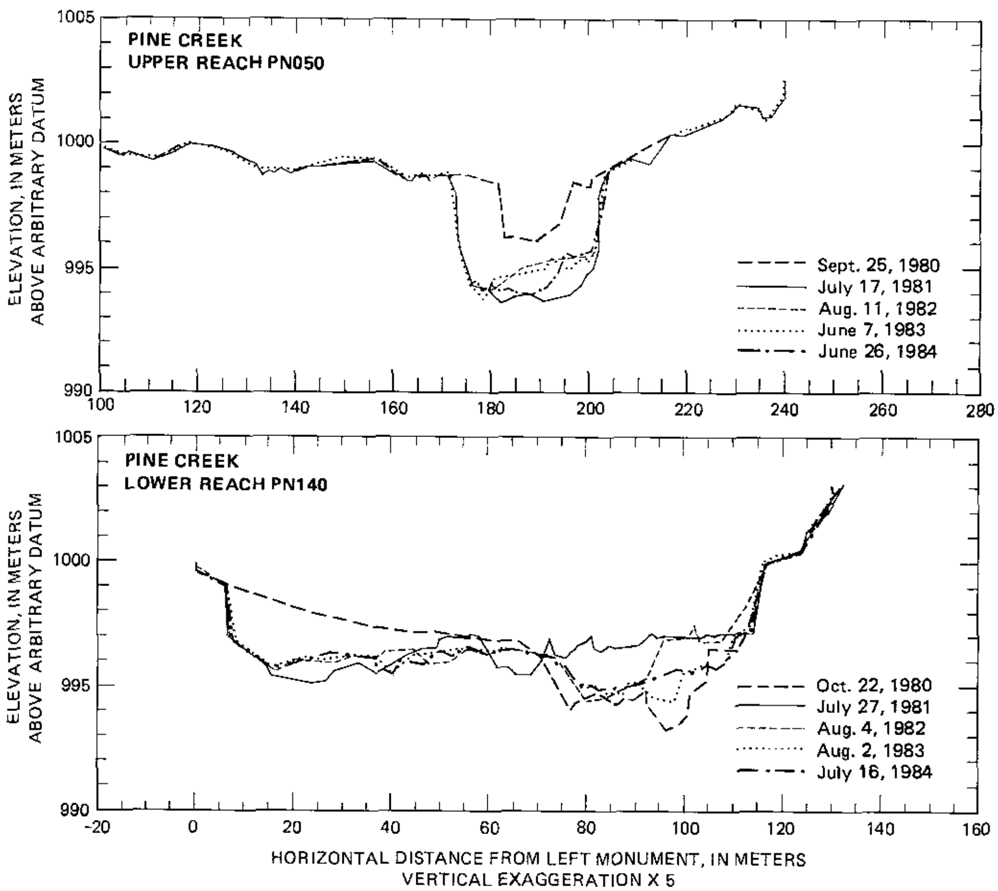

FIGURE 4. – Surveyed channel cross-section profiles showing post-eruption channel adjustments, Pine Creek, Washington. Upper reach, PN050, is 16.3 kilometers above mouth; lower reach, PN140, is 4.8 kilometers above mouth.

slight increases in width in upstream reaches. Adjustments during WY 1982-84 in downstream reaches were dominated by channel shift, incision, and enlargement; the channel narrowed or widened only slightly (fig. 4).

Changes in mean bed elevation, channel width, pattern, and sinuosity were largest during WY 1981, the first year following the eruption. The channel, as defined by the lateral limits of erosion, widened an average of 19 m at the measured cross sections (fig. 5). Widening ranged from 3 to 63 m, or from 10 to 305 percent. Erosion of the channel along the upper 4.2 km of the study reach lowered mean bed elevation by an average of 2.5 m; the incision at measured cross sections ranged from 1.2 to 4.8 m (fig. 5). Aggradation along the lower 6.9 km of channel raised mean bed elevations between 0.1 and 2.5 m, or by an average of about 1.0 m. Channel sinuosity decreased by an average of three percent in downstream study reaches.

Changes in channel width and mean bed elevation during WY 1982 were substantial, particularly in downstream reaches, but they were not nearly as large as the changes that occurred during WY 1981 (fig. 5). The channel

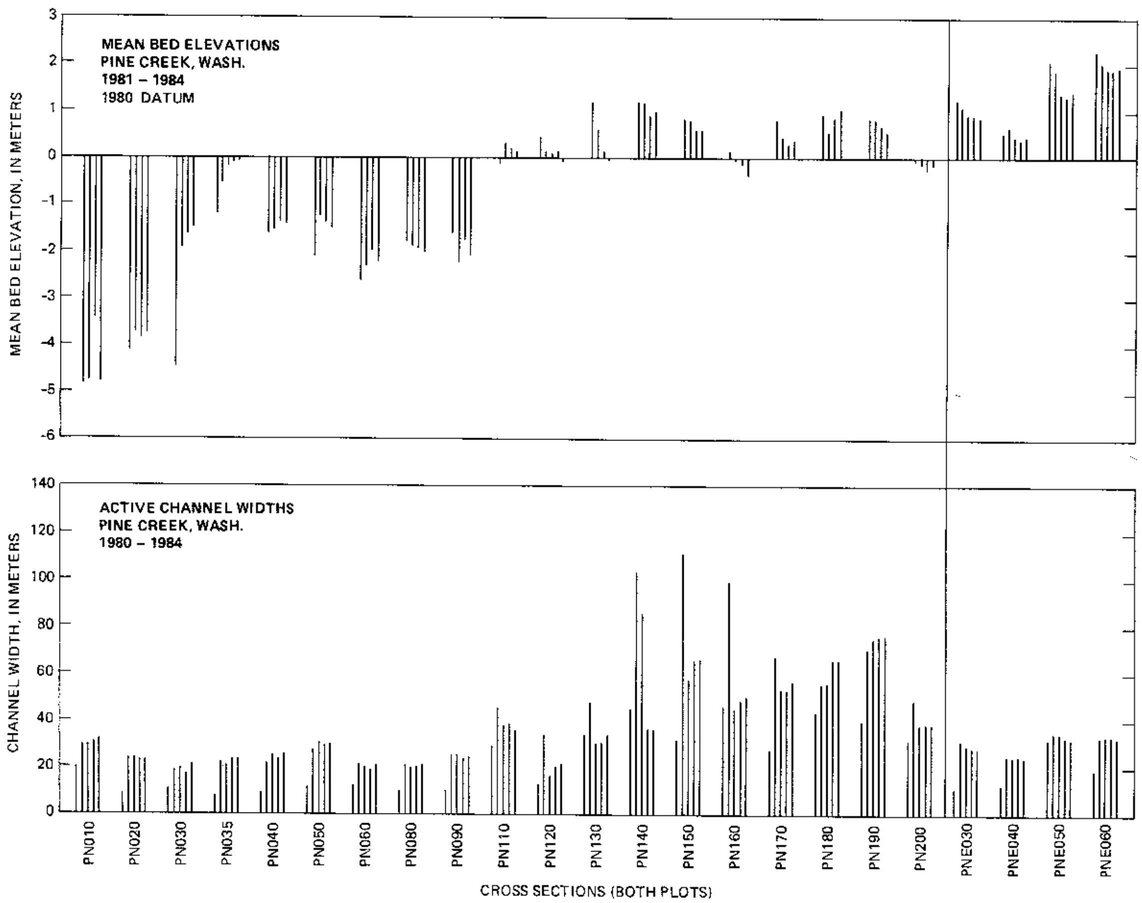

FIGURE 5 - Mean bed elevations and active channel widths at measured cross sections, Pine Creek, 1980-1984. Consecutive bars for each cross section represent consecutive annual measurements beginning in the year indicated. Locations of the cross sections are shown in figure 1.

widened at one third of the measured cross sections, but rates of widening were only 1 to 20 percent of those the previous year. Changes in mean bed elevation during WY 1982 were commonly the reverse of those observed during WY 1981 (fig. 5). Upstream reaches that had incised during WY 1981 aggraded and widened slightly. Downstream reaches that had aggraded and braided during WY 1981 channelized as the river incised into alluvium deposited the previous year. Consequently, channel width decreased at most downstream sections during WY 1982 (fig. 5). By the end of WY 1982, the channel at five of the seven downstream cross sections had assumed a position and geometry that were roughly maintained during WY 1983 and 1984.

During WY 1983, channel shift and planform modification occurred locally along downstream reaches, where the channel incised and enlarged and (or) widened slightly (figs. 4-6). Local erosion or deposition occurred in upstream reaches (fig. 4).

The general shape and location of the 1983 channel was maintained during WY 1984, with local aggradation or degradation of the bed at some cross sections, but negligible widening or enlargement of the channel (figs. 4-6). Increases in cross-sectional area caused by local erosion of the bed were offset by decreases in area caused by deposition elsewhere in the cross section. Therefore, changes in mean bed elevation were smaller than those that occurred in previous years (fig. 5).

In summary, post-eruption streamflow has altered the size, shape, and pattern of the lahar-affected channel. The channel has widened, enlarged, and straightened somewhat. Fluvial processes modified the lahar-produced U-shaped channel to one that is trapezoidal in shape with moderately sloping banks along much of its length (fig. 4). Channel roughness and stability were increased as progressive erosion developed a surficial layer of cobble-sized material along the streambed and banks.

DISCUSSION

The aggradation and braiding of the channel that occurred in downstream reaches where the valley widens and the slope decreases was expected, given the volume of sediment eroded from upstream reaches during the first year following the eruption. Terraces and stream banks provided local sources of sediment during channel widening. However, these local sources do not account for the total volume of material deposited at downstream cross sections. Material derived from upstream sources played an important role in downstream channel aggradation and braiding. Subsequent incision and reduction in braiding in downstream reaches resulted in part from the marked decrease in sediment supplied from upstream channel erosion. Channel degradation in downstream reaches during WY 1982 was aided by the occurrence of the largest flow of record (118 m^3/s) on February 20, 1982, which eroded large volumes of sediment.

The extensive incision and widening upstream between 1980 and 1984 was in part a response to the lahar-induced modifications to the channel and drainage network. These modifications include steepening, straightening, rerouting, and channelization of flow. Upstream progressive degradation (Galay, 1983) was a dominant process as adjustments in channel gradient

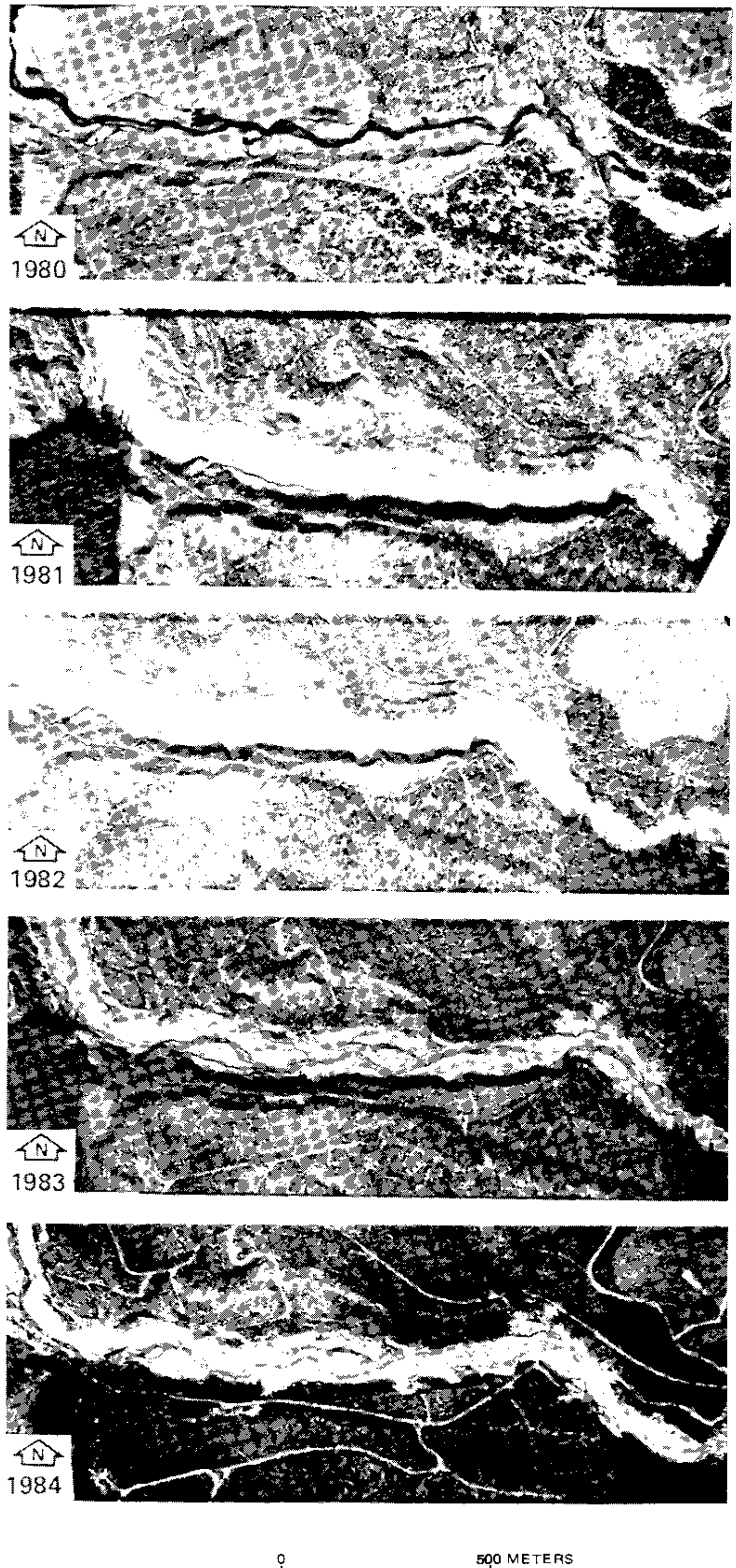

FIGURE 6. – Aerial photographs for the reach of lower Pine Creek containing cross sections PN140–PN170, showing channel planform changes through time.

occurred following steepening and straightening of the channel by the lahar. Smoothing of the knickpoint-stepped profile and channel incision within the narrow, vertical-walled trench to which the channel was confined occurred rapidly with the first winter storm following the 1980 eruption. Streamflow undermined the banks, causing bank failure and widening the channel.

Other factors may have affected the magnitude and timing of channel adjustments: 1) heavy rainfall during November 1980, on upper parts of the drainage basin that usually are covered by snow, 2) hydrologic disturbance of upper parts of the drainage basin from airfall deposits, resulting in increased volume of sediment-laden runoff to the channel, 3) increased transporting capacities of flows due to high concentrations of suspended sediment (Simons and others, 1963), and 4) the magnitude and timing of post-eruption peak flows.

Peak flows measured near the mouth of Pine Creek were similar during WY 1983 and 1984. Streamflow during WY 1984 caused little change in size, shape, location, and pattern of the channel. Future modifications of the channel probably will be discharge dependent, with a gradual return of the channel to its pre-eruption morphology.

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