SECTION 15

TREND ASSESSMENT

Page

SHORELINE EROSION PROCESS, LAKE SAKAKAWEA, ND

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ABSTRACT

Detailed measurement of 20 stations along the east end of Lake Sakakawea since 1983 has provided more shoreline erosion data than ever before for an inland lake. Additional data from sediment rangeline surveys in 1969, 1979, and 1988 have provided an unique opportunity to define erosion processes and attempt a predictive equation. Approximately 78% of the total recession is the result of wave erosion; the remaining 22% is due to frost thaw failure. Recession is strongly dependent upon lake levels, with the predicted average monthly recession during high lake levels being equal to $-46 + 1.20\sqrt{A} + 0.02\sqrt{B} + 0.10\sqrt{C} - 0.70\sqrt{D}$, where A = the effective fetch at a given station, B = the bank height, C = the percentage of beach sediment, pebble size, and D = the beach slope. The predicted average monthly recession during low lake levels = $0.142 + 0.004B + 0.001G$, where G = the orientation of the bank with respect to the sun. Goodness of fit for the two equations is 46% and 35%, respectively.

INTRODUCTION

Lake Sakakawea is located in west central North Dakota (figure 1) and operated by the U. S. Army Corps of Engineers, Omaha District. With a length of nearly 200 miles, a depth of 180 feet near the dam, and widths over most of the lake varying from 3 to 5-miles, it is one of the largest man-made lakes in the U. S. It has nearly 1500 miles of shoreline, of which at least 87% is eroding. Annual rates of erosion average about 3.4 ft/yr, but during high lake levels recession at individual stations has ranged up to 26 ft.

Damage from erosion to private property and adverse impacts at government recreational facilities are of increasing concern. Other impacts of consequence include destruction of wildlife habitat and fish spawning areas, damage and loss of archeological sites, and deterioration of lake water quality.

Conventional techniques for analysis of erosion at Lake Sakakawea and other Corps lakes on the Missouri River mainstem were determined (Cordero, 1982) to be substantially inadequate. This recognition led to a cooperative agreement in 1983 between the Corps and the University of North Dakota to research the magnitude and causes of bank recession and to develop new predictive capability. The University had recently completed a shoreline erosion study on Lake Orwell in Michigan for the U. S. Army Corps of Engineers Cold Regions Research Laboratory. The knowledge from that study plus the University's close proximity to Lake Sakakawea made the University ideally suited to undertake the new study. Dr. John R. Reid served as the principal investigator for both studies.

The findings from the investigation discounted preliminary thoughts that lake ice might be a major factor in the erosion process. They also revealed that approximately 78% of the total recession stemmed from wave erosion during normal to high pool levels. And, they led to the discovery that 22% of the bank loss resulted directly and indirectly from frost thaw failure.

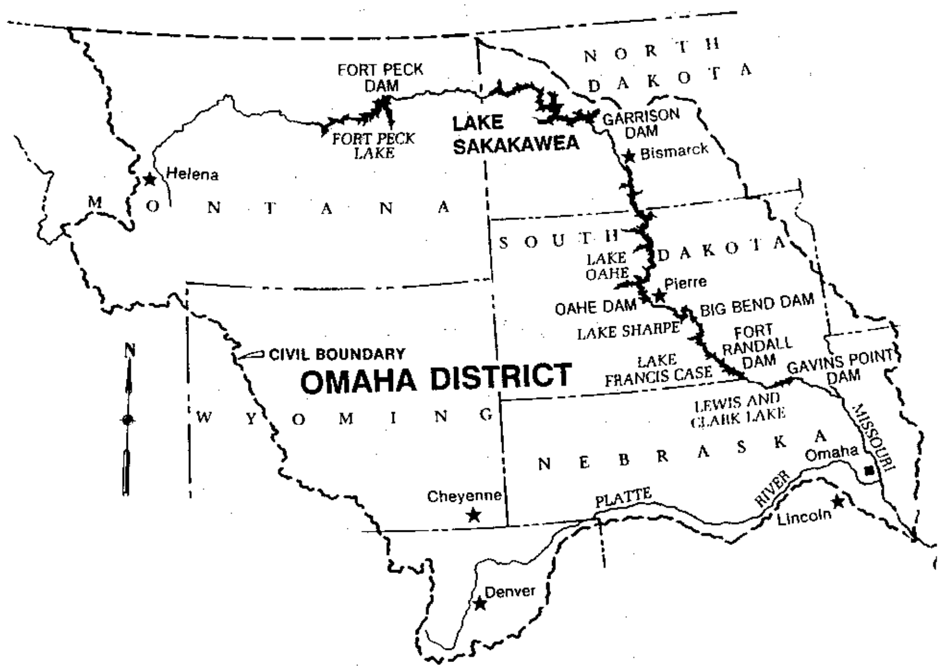

Fig. 1. Generalized map of Omaha District showing location of Lake Sakakawea.

BACKGROUND

Lake Sakakawea was formed by the construction of Garrison Dam, located about 65 miles upstream of Bismarck, North Dakota. Construction began in 1947 and closure was made in April 1953. A full operating level was first achieved in 1965. Pool levels vary seasonally by some 10 to 25 feet, with minimum elevations in the winter and maximum elevations in mid-summer. The lake is completely ice covered for three to four months each winter, usually from early January to mid April.

Banks within the project area, which is limited mostly to the 9-mile reach of the lake immediately adjacent to and west of the dam, vary in height from about 6 to 82 feet. The upper portions are generally comprised of loess overlying glacial tills, which in turn, particularly in the higher banks, overlie lignite outcroppings interbedded with sandstone, mudstone, and occasional clinker (scoria). Surface topography is grass-covered, rolling uplands.

The erosion cycle begins in the late winter as thaw from the winter frost causes mass failure of the upper bank. Colluvium from this source then builds up at the toe of the banks, which in turn is eroded by storm-wave action during higher pool levels. If the colluvium is eroded away or overtopped by wave wash or high lake levels, the waves come in contact with the primary sediment or bedrock, thus causing the banks to be undercut and a new process of erosion to begin. For many of the high bank locations the effect of the undercutting is to set into motion the development of extensional cracks at the tops of the bank. These, in turn, expand in width until bank failure releases the stress and another process of erosion begins.

Recession rates within the study area were not found to be substantially different from rates at other lakes on Missouri River. Differences primarily reflect variations in lake orientations, operating procedures, local geologic conditions, and measurement practices. Together the six lakes have nearly 5,940 miles of shoreline along 745 miles of the river. Ft. Peck Lake, located upstream of Lake Sakakawea, is estimated to have an average annual rate recession of 3.9 ft/yr. Lake Oahe, the next downstream from Lake Sakakawea, averages 6.6 ft/yr; Lake Sharpe averages 8.2 ft/yr, Lake Francis Case is 3.2 ft/yr, and at Lewis and Clark Lake the average is 2.3 ft/yr. Recession rates in the 10 to 15 ft/yr range exist at some sites on six of the lakes. Lake Sharpe, the only one of the six with a steady pool level, has two stations with excessive erosion, one with 30 ft/yr and the other with 40 ft/yr.

CONVENTIONAL METHODOLOGY

Erosion assessments for lakes on the Missouri River have traditionally made use of the "template method" illustrated in figure 2. It relied heavily on available topographic mapping, sediment range line hydrographic survey data, pool level data, and information on the significant design wave height. Consideration was also given to bank material characteristics when such information was available.

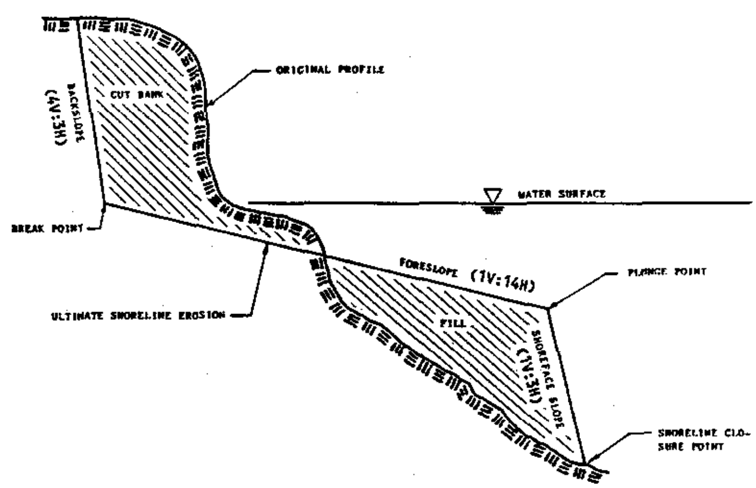

Fig. 2 Template for predicting ultimate shoreline erosion recession (from Cordero, 1982).

In using the template the assumption was made that bank recession would occur until the cross-sectional area eroded equaled the cross-sectional area of fill under a stabilized beach profile (assumed to have a slope of 1 on 14). Adjustments for each different site were made in the cut-fill balance to reflect local conditions. The slopes identified in figure 2 reflect averages derived from field observations during the early years following construction of the mainstem lakes. The break point was assumed to reside at the 80% pool exceedence level plus a significant design wave height. In selecting this level, the assumption was made that higher pool levels would occur so infrequently and for such short duration as to not have a significant effect on the erosion and beach stability process.

Field observations in more recent years have found the template method to be well suited for banks containing a high percentage of sands, but totally inadequate for those comprised mostly of silts and clays, since the finer materials go into suspension and move away from the site without remaining to form a stabilized beach area.

DATA COLLECTION ACTIVITIES AND GENERAL OBSERVATIONS

Detailed measurement stations were established along the eastern end of Lake Sakakawea at the locations illustrated in figure 3. The goal in selecting the stations was to have sites easily accessible from land and located in areas that represented erosion under a variety of wind and solar exposures and bank material characteristics. Bank recession pins served as the principal means for measuring bank erosion. These were sets of 6-inch nails placed 15 and 30 feet back from the top bank edge and tagged for easy location. Initially, each station also included a set of 12-inch long spikes inserted normal to the bank face to observe erosion by rainsplash and overflow. However, these were soon abandoned once it was confirmed that rainfall erosion was not a key factor in the bank recession process. Measurements of beach and bank profiles were measured using a Brunton compass and the offshore profiles were measured with a sonar recorder and stadia readings. Colluvium accumulations were determined following the end of each spring. In addition, frost tubes were installed at five of the stations to monitor frost depth penetration and a thermograph was located near one of the stations to monitor temperature through the winter, thus enabling recordings of the freeze-thaw cycle.

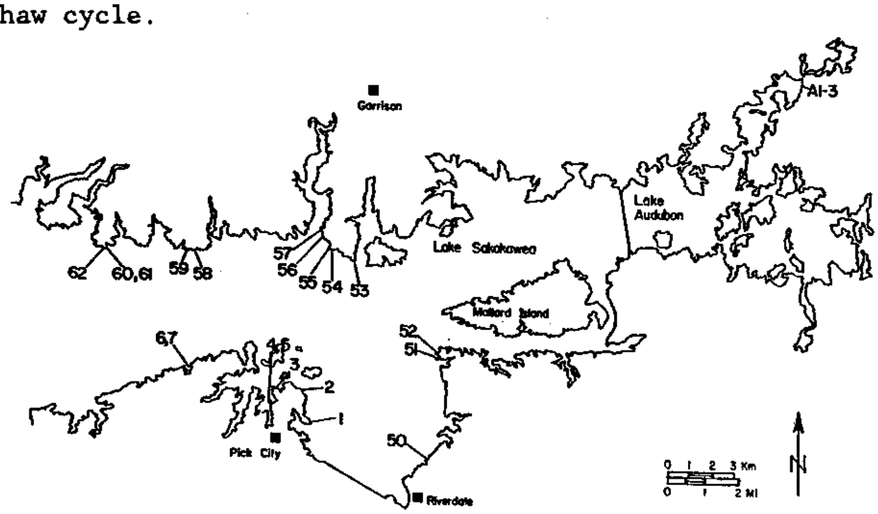

Fig. 3. Measurement stations.

The complete data set included observations of pool levels, storm events, bank recession, colluvium volumes, bank lithology and stratigraphy, beach and bank profiles, offshore profiles, orientation of the bank with respect to prevailing wind directions, frost depths and thermograph readings, percentages of coarse beach clasts, and tension cracks along the top of the banks. Early phases of the investigations also included observations of overland erosion, precipitation, rainfall intensity, and piezometer well readings, but these were soon abandoned because of difficulty in determining correlation coefficients. In doing so, however, it was recognized that rainfall-induced erosion may eventually become important in the recession process as the banks gradually stabilize against storm-wave attack. Ground water discharge (seepage) also was discounted as a major factor since "the lake recharges the regional ground water table, which lies below lake level (Croft, 1985)." Detailed discussions of all parameters are available in the literature. Those parameters selected for regression analysis are discussed below:

a. Lake levels: High lake levels occurred only 12 months during the 1983 to 1988 study but are associated with 78% of the total measured erosion occurring during this interval. The lake (pool) level-bank recession relationship in Figure 4 vividly illustrates the significance of high lake levels and the strong correlation with major increases in average bank recessions rates. High lake levels refers to instances in which the lake exceeded for an extended period a critical elevation of 1846 feet m.s.l., the point at which erosion became most noticeable. The relatively low recession rates observed during 1983 occurred because the preceding three years had been characterized by low levels, and large accumulations of colluvium protected the base of the bank. The wave erosion was mainly limited to removing some of that colluvium.

b. Bank Height: Bank recession during the colder season revealed stronger correlation with bank height than did wave-induced bank recession during the warmer months. The amount of colluvium found at the base of the high banks at the end of the colder season was found to be several magnitudes greater than that observed for much lower banks, while during the warmer months only a slight difference was detected. Generally speaking, it was observed that low banks respond quickly to wave undercutting whereas high banks respond slower and with lesser magnitude.

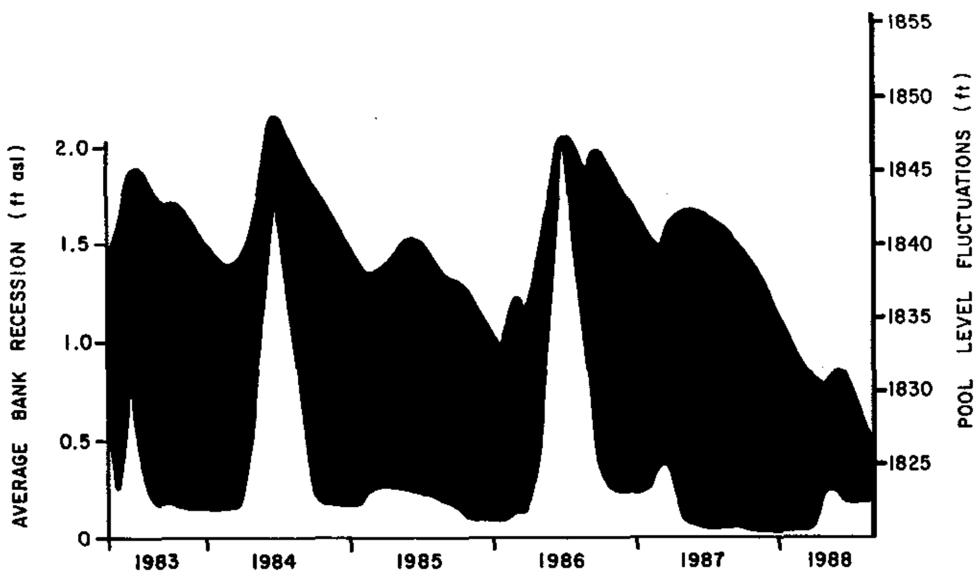

Fig. 4. Lake (Pool) Level-Bank Recession Relationship.

c. Effective Fetch: This is the effective distance over water from which wave energy can be expected. It is derived from a procedure developed by Saville (1954) and can generally be assumed to be less than the greatest straight line distance over open water in areas where irregular boundaries exist. It recognizes that waves are generated from the direction of the wind and attack banks at some considerable angle to it (Reid, 1986).

d. Beach Composition: This refers to percentages of beach area covered by clasts of pebble size or larger. Correlations of percentages of coarse clast with bank recession rates revealed that a relationship exists between beach armoring and rates of recession. Presumably, coarse beach clast and other forms of armoring or natural rip-rap (i.e., lag concentrations of glacial boulders, sandstone masses, and petrified logs) effectively impede wave erosion before they become submerged by rising lake levels (Millsop, 1985).

e. Beach Width: Wide beaches correlate with stable banks, all other factors being equal. To offset the influence of variable lake levels during the field measurements, the beach widths were always measured from the bank profile at the same elevation.

f. Offshore Slope: Flatter offshore slopes appear to correlate with beach stability and reduced bank recession rates. The slope measurements were derived from continuous fathometric recordings tied directly to the bank profile stations.

g. Orientation of Bank With Respect to Dominant Wind Direction. Banks facing open water wind directions from the north, northwest, and southwest were found to have higher rates of bank recession than those facing east and southeast directions. Those facing the southwest had recession rates nearly double those from the north and northwest. It was observed that banks that face into the storm winds, that are composed of highly jointed rock or sediment, or that have sand and gravel beds near wave level, experienced the greatest wave erosion.

h. Orientation of Bank With Respect to Sun. Banks facing northerly directions were found to hold moisture longer than those exposed to solar radiation and thus had a stronger tendency to undergo frost rupture and subsequent thaw failure. Such failure is estimated to be directly or indirectly responsible for about 22% of the total bank recession along Lake Sakakawea. When thaw occurs, these northerly facing banks develop cracks parallel to the free face of the exposed bank. Although the cracks develop through the year, the majority first appear in late winter as a result of either cold contraction or tension caused by the open face. Snow and ice accumulate in these cracks and cause further weakening of the block created by the crack. The cold season failure, then, involves the flowing and sliding of muds and the sliding, slumping, and toppling of large blocks. The block failure continues intermittently through the year.

REGRESSION ANALYSIS

The differences observed between the summer and the cold season recession rates led to development of two separate predictive equations: one reflects factors associated with storm-wave erosion during the warmer months, and the other takes into account the bank failure stemming from solar influences on freeze-thaw processes. The resulting seasonal equations are:

$$Rs = 2.75 + 0.11\sqrt{A} - (0.11\sqrt{B} + 0.04\sqrt{C} + 0.44\sqrt{D} + 0.19\sqrt{E} + 0.12\sqrt{F})$$

and $$Rw = 0.131 + 0.004B - 0.001G,$$

where Rs is the predicted monthly bank recession (ft) during the warm season, Rw is the predicted monthly bank recession (ft) during the colder season, A is the effective fetch (miles), B is the bank height (ft), C is the percentage of beach clasts smaller than pebble size, D is the angle of the offshore slope (degrees), E is the sine of the angle between the beach face and the dominant wind direction, F is the beach width (ft), and G is the angle between the bank orientation and the direction of least influence on freeze-thaw rates (southeast). The F-ratio for Rs was found to exceed the 75 percent confidence level and have a Goodness of Fit of 58%. For Rw, the F-ratio exceeded the 97.5 percent confidence level and had a Goodness of Fit of 39%. The yearly recession rate (Rt) is the sum of Rs and Rw multiplied by 6 months.

Since the equations above do not take into account pool level effects, the parameters were further analyzed into a second set of equations to reflect recession rates for high and low pool levels. These equations are:

$$Rh = -0.46 + 1.20\sqrt{A} - 0.02\sqrt{B} + 0.10\sqrt{C} - 0.70\sqrt{D}$$

and $$Rl = 0.142 + 0.004B - 0.001G,$$

in which Rh is the predicted monthly bank recession during high pool elevation months (ft), Rl is the predicted value during lower pool level months (ft), and A, B, C, D, and G are as defined above. The parameters E and F were eliminated from this second set of equations since E, the exposure direction, was concluded to be a duplication of the fetch factor, and F, the beach width, was concluded to be a duplication of the beach slope factor. The F-ratio for Rh was found to exceed the 75 percent confidence level and have a Goofness of Fit of 46%. For Rl, the F-ratio exceeded the 95 percent confidence level and had a Goodness of Fit of 35% While the statistical significance of the these two equations is lower than for the previous two equations, they offer the advantage of being applicable to other locations such as the Corps' sediment rangeline stations where the measured data can provide some limited means of verification.

The total predicted recession (Rt) over a period is the sum of these two equations, with each multiplied by the appropriate number of months of high and low pool-levels predicted. In equation form this becomes:

$$Rt = H(Rh) + L(Rl),$$

where H is the number of predicted high pool-level months for the period of interest and L is the predicted low pool-level months.

CONCLUSION

Each future year of additional data will enhance the significance of the predictive equations and with added data may make it possible to include other variables such as lithology and structure factors. Not surprisingly, higher recession rates during the investigation were found to correlate best with banks located at headlands, and the lowest rates with stations located in areas in protected bays with low topography. Of the forty two parameters taken into consideration during the course of the study, those recognized as being most significant to the erosion process included effective fetch, bank height, beach composition, orientation of the shore with respect to prevailing winds, offshore slopes and beach width. For the winter months, the rates correlated best with bank height and orientation of the banks with respect to the sun. While rainfall erosion, groundwater seepage, and ice shove were recognized as insignificant for the present study, their importance could change significantly for other lakes under differing conditions. Rainfall erosion may become the dominating factor for bank loss at all locations as they gradually become stabilized against freeze-thaw failure and storm-wave attack.

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SEDIMENTATION IN LAKE FRANCIS CASE 1953-1986

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ABSTRACT

The Omaha District of the U.S. Army Corps of Engineers has been collecting sediment related data at Lake Francis Case, as well as the White River, in South Dakota since 1953. Data collected include cross sections, bed material samples, sediment deposit densities, and suspended sediment inflow. The sediment brought into Lake Francis Case is composed of fine clay with a predominance of the mineral montmorillonite. The mechanisms for erosion, transport, deposition, and consolidation of such cohesive sediments are very different from those for non-cohesive sediments with which most engineers are familiar. An annotated record of basic observations, tabulations, and geomorphic trends at the lake are presented.

INTRODUCTION

The Fort Randall Dam - Lake Francis Case Project is located on the Missouri River near Lake Andes, South Dakota, at river mile 879.48 as shown in Figure 1. The lake is the eighth largest impoundment in the United States, with a storage capacity of 5,940,000 acre-feet, a surface area of 102,037 acres, and a length of 108 miles at its maximum operating pool level (1375 feet MSL). Lake width varies from one to over three miles, and total shoreline perimeter is in excess of 540 miles. Impoundment of water in the lake began in the latter part of 1952. In 1953 the project was operated in conjunction with the Fort Peck project to provide releases to meet navigation flow requirements below Fort Randall. By the latter part of 1953 and in 1954 the project began operating in conjunction with Fort Peck and Garrison Reservoirs. The remaining three Missouri River Main Stem Dams, Gavins Point, Oahe, and Big Bend, were completed in 1955, 1958, and 1963, respectively. All six reservoirs are operated in concert to provide control of flood flows and to regulate available flow for production of hydroelectric power, navigation, and maintenance of sufficient flows for domestic and industrial uses and for water quality control purposes. Location, drainage area, and date of closure for each project are shown in Table 1.

TABLE 1
MISSOURI RIVER MAIN STEM RESERVOIRS
DRAINAGE AREA AND DATE OF CLOSURE

Project	1960 River Mile	Drainage Area (sq miles)	Date of Closure
Fort Peck Dam - Fort Peck Lake	1771.5	57,500	June 1937
Garrison Dam - Lake Sakakawea	1389.9	181,400	April 1953
Oahe Dam - Lake Oahe	1072.3	243,500	August 1958
Big Bend Dam - Lake Sharpe	987.4	249,300	July 1963
Fort Randall Dam - Lake Francis Case	879.5	263,500	July 1952
Gavins Point Dam - Lewis and Clark Lake	811.1	279,500	July 1955

The Omaha District has maintained an extensive data collection program at Lake Francis Case (as well as the remaining District reservoirs). Data collected include cross sections, bed material samples, suspended sediment samples, and deposit densities. In addition to the Corps data collection, the U.S. Geological Survey operates a suspended sediment load and water discharge gage on the White River Arm of the lake near Oacoma, South Dakota. This report presents an annotated record of basic observations and tabulations of the Omaha District's historic field data collection at Lake Francis Case and the White River, and documents pertinent geomorphic data and trends. Detailed results are found in MRD Sediment Memoranda No. 7, "Lake Francis Case Aggradation Study (Including White River), 1953-1986", May 1989.

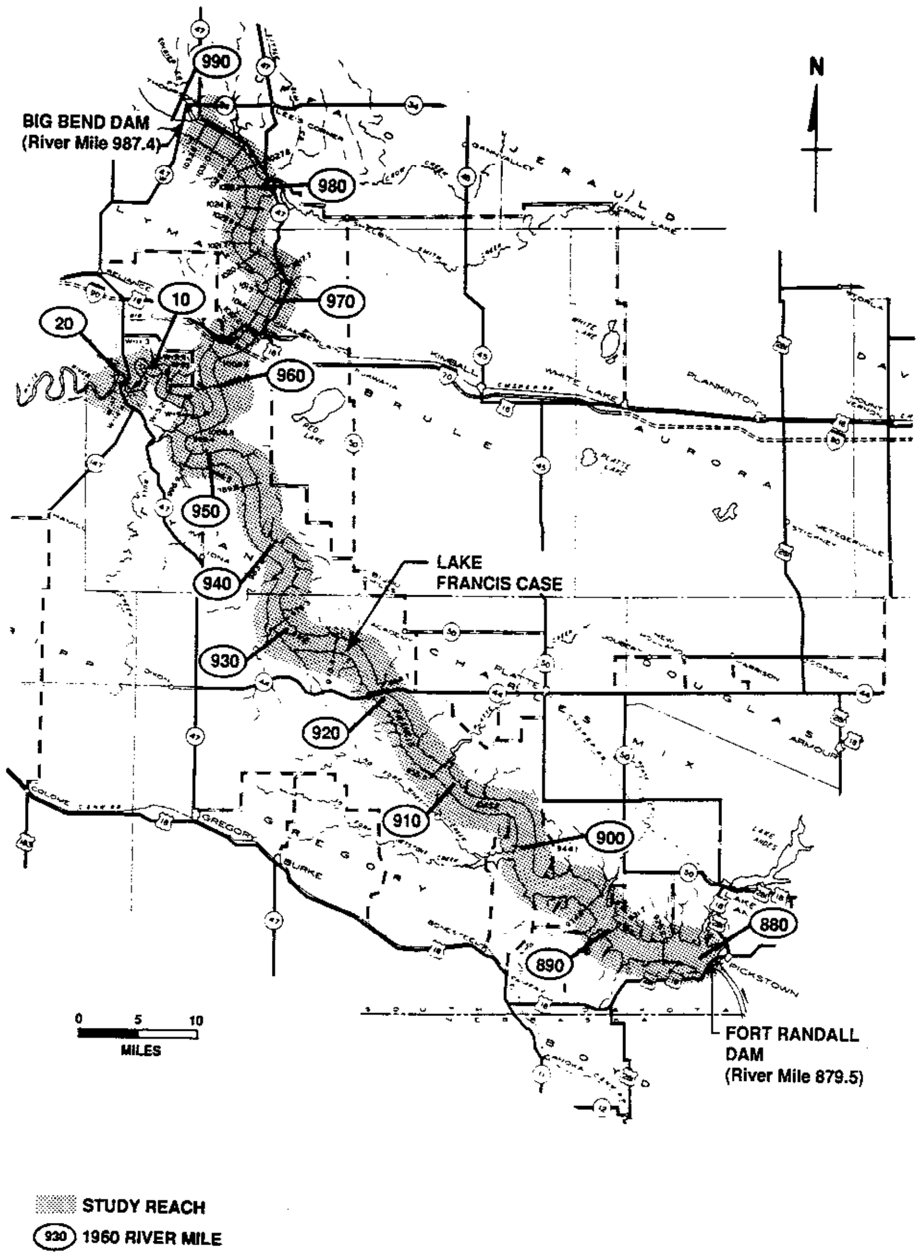


Figure 1. Study Location Map
SEDIMENT INFLOW PROCESSES

The three elements of sedimentation processes investigated for this report are bed sediment characteristics, sediment inflow rates, and the density of sediment deposits.

Bed Sample Analysis

Between 1953 and 1962, a regular bed sediment sampling program was conducted throughout the area then bounded by Lake Francis Case. This included the 45 mile reach upstream of river mile 987.4 which is now inundated by Lake Sharpe. Since 1962, the only bed sediment sampling was completed in 1981.

Typically between 4 and 8 samples were taken across each section in any survey. Each of these samples was graded (by sieve analysis) and composite or average sediment sizes were determined for specific percent passing value. In general, samples were obtained between May and September in any given year.

Figure 2 is a plot of the D50 grain size versus river mile, for the reach upstream of the White River confluence. This curve, representative of all grain sizes for the lake, shows significant fluctuations in specific values of sediment size, with no apparent historic trend. When viewed as a continuum, the grain size appears to be relatively constant. Although not presented herein, the largest size bed sediment materials can be observed in the White River which probably reflects higher flow velocities.

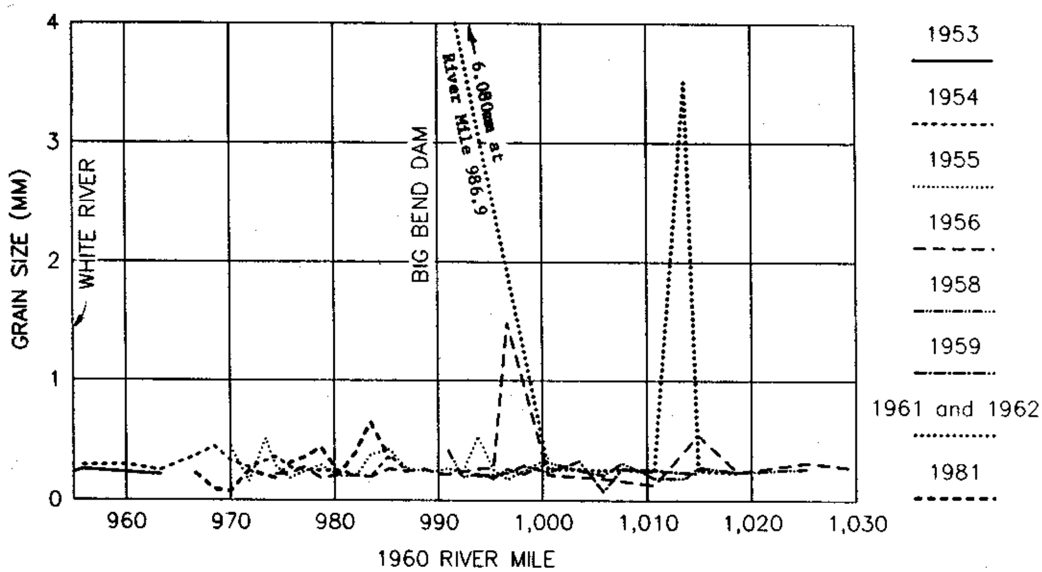


Figure. 2 Reservoir Bed Materials (D50 Grain Size)

Sediment Inflow

Practically all sediment materials that are delivered to the lake are river bourn from either the Missouri or White Rivers. The exception to these sediment sources are local overland runoff and shoreline erosion of the reservoir banks. Historically, the primary influence on the rate of sediment inflow into the reservoir has been the timing and extent of sediment contribution areas being removed by the construction of the main stem dams. While the impacts of the Fort Peck project were established long before the inception of the Fort Randall project, the remaining three upstream projects have had the progressive effect of reducing the sediment contributing drainage area from 205,980 square miles to 14,150 square miles, and the estimated sediment inflow from 76 million tons in 1953 to about an average of 9.5 million to 1986. Currently most of the sediment inflow into Lake Francis Case is contributed by the White River drainage area of 10,200 square miles.

Table 2 shows the measured sediment inflow into Lake Francis Case (from U.S.G.S. and Omaha District records) corresponding to the hydrographic survey intervals. Also shown is the measured average annual sediment load for these time increments. The progressive decline in average annual load rates can be seen in the table with a peak average of about 46 million tons per year in the 1953 to 1957 period to the current rate of 9.5 million tons per year.

TABLE 2
MEASURED SEDIMENT INFLOW
ACCUMULATED BETWEEN HYDROGRAPHIC SURVEYS

Hydrographic Survey Date	Time Interval (years)	(1) Total Measured Sediment Load (tons)	Average Annual Measured Sediment Load (tons/year)
1 Jun 1953 - 1 Aug 1957	4.17	187,707,062	45,013,684
1 Aug 1957 - 1 Aug 1962	5.00	108,472,171	21,694,434
1 Aug 1962 - 1 Jun 1967	4.88	56,599,431	11,598,244
1 Jun 1967 - 15 Jul 1973	6.08	65,664,285	10,800,047
15 Jul 1973 - 1 Jun 1977	3.96	24,409,218	6,163,944
1 Jun 1977 - 15 Jul 1981	4.04	38,128,857	9,437,836
15 Jul 1981 - 15 Aug 1986	5.08	49,260,942	9,697,036

(1) For period 1 Jun 1953 - 1 Aug 1962, sediment sampled on Missouri River at Pierre, SD; Bad River at Fort Pierre, SD; and White River at Oacoma, SD. For period following 1 Aug 1962, sediment sampled on White River only.

The sediment contributed by the White River is primarily montmorillonite clay (with lesser amounts of illite, chlorite, and kaolinite) and its physical characteristics are highly sensitive to water chemistry and temperature. As these sediments react with ions in the water, they form larger agglomerated particles, which, as they settle, have a tendency to form a very loose fluff with very little resistance to shear stress. These sediments in this state can be transported as a density current or by being subjected to very low shear stresses induced by moving water. Field experience shows that this material is easily resuspended and redeposited downstream during peak flow releases from Lake Sharpe, located 30 miles upstream. With time, this fluff tends to break down and consolidation occurs. The material then develops a relatively high resistance to erosion.

Density of Sediment Deposits

Density of sediment deposits are presented as Table 4. Density probe observations were made during 1962, 1967, and 1973. The average density of deposits in Lake Francis Case (including the White River Arm) is about 32 pounds per cubic foot, dry weight. Densities estimated from the ratio of measured sediment inflow to measured sediment depletion show an average value of about 35 pounds per cubic foot. The average observed deposits density in the White River Arm varies with time from about 25 to 55 pounds per cubic foot dry weight.

TABLE 4
DENSITY OF SEDIMENT DEPOSITS

				Average Density (lbs/cubic foot)							
				Wet Density				Dry Density			
		Range Boundary	Verticals	Percent of Deposit Depth				Percent of Deposit Depth			
Location	Year	(1960 Rivermiles)	Measured	0-25	25-50	50-75	75-100	0-25	25-50	50-75	75-100
Lake Francis Case - Missouri River											
	1962	926.2 - 1032.6 (884.05 - 986.90)	17	77	82	84	89	24	31	35	43
	1967	934.6 - 1005.2 (892.00 - 960.03)	19	73	78	81	82	17	25	30	32
	1973	970.4-1012.2 (926.49 - 966.66)	15	75	81	84	88	21	30	35	42
Lake Francis Case - White River Arm											
	1962	W-3.9 (3.9)	1	73	76	82	95	17	22	32	52
	1967	W-1.9 (1.9)	1	85	92	84	105	20	47	35	69
	1973	W-0.0 (0.0)	1	85	98	-	-	36	57	-	-

RANGE CROSS SECTIONS

Thirty-four range sections have been established for the Missouri River between Fort Randall Dam and Big Bend Dam, and an additional 14 ranges on the White River, at the locations shown on Figures 1 and 7. Data for the sections typically extend across the entire valley from about elevation 1400 feet MSL on both sides. Cross section plots for all 48 ranges would be too voluminous for this report, therefore, four sections, deemed to be representative of observed deposition patterns, were chosen and presented as Figures 3 through 6. (Note that cross section locations are denoted by their 1941 river mileage.)

The typical distribution of sediments across the section is relatively uniform with possibly slightly deeper deposit depths in the original stream channel area. The average depth of deposition gradually rises from about 4 to 6 feet at Fort Randall Dam to about 10 feet at river mile 912. The rate of depth increase then steepens all the way to the confluence of the White River at river mile 954.3, with average deposition depths of about 25 feet and depositions depths greater than 40 feet in the vicinity of the original channel. Downstream from river mile 932 deposition takes place mainly in the old river channel, and to a lesser extent on the overbanks (represented by Figure 3). Between river miles 932 and 960, the deposits have covered the entire valley and have abolished the topographic rise and fall of the bottom lands to form a new flat lake bottom (represented by Figure 4). From river mile 955 upstream, the deposition depth rapidly diminishes to an average of about 10 feet at mile 967, with deposition consisting mainly of filling the original stream channel from that point to upstream (represented by Figure 5).

Cross section changes on the White River are characterized by average deposition depths greater than 25 feet at the mouth. This depth decreases to a distance of about 8 miles upstream where only the original stream channel is being filled. Filling of the original channel continues to 14 miles upstream, after which slight channel erosion is observed. A further observation for White River range section plots is that the river has formed a considerably smaller channel on the deposition to carry the normal river flow (as represented by Figure 6).

Figure 3. Cross Section 964.7 (River Mile 921.07)

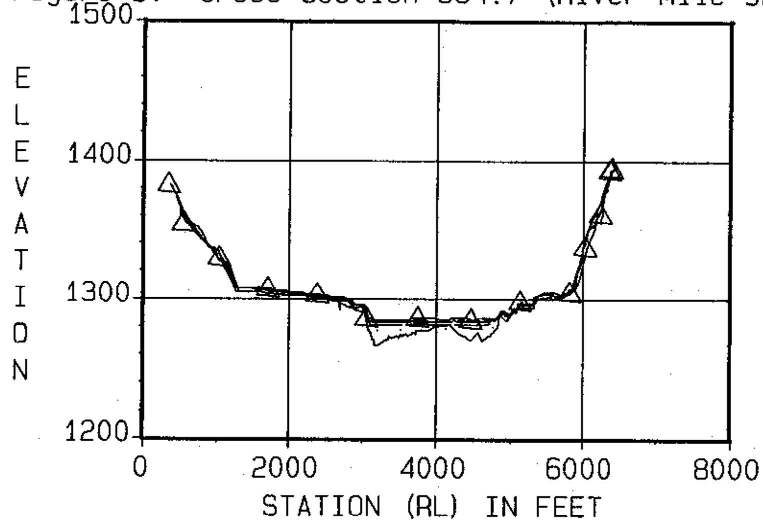
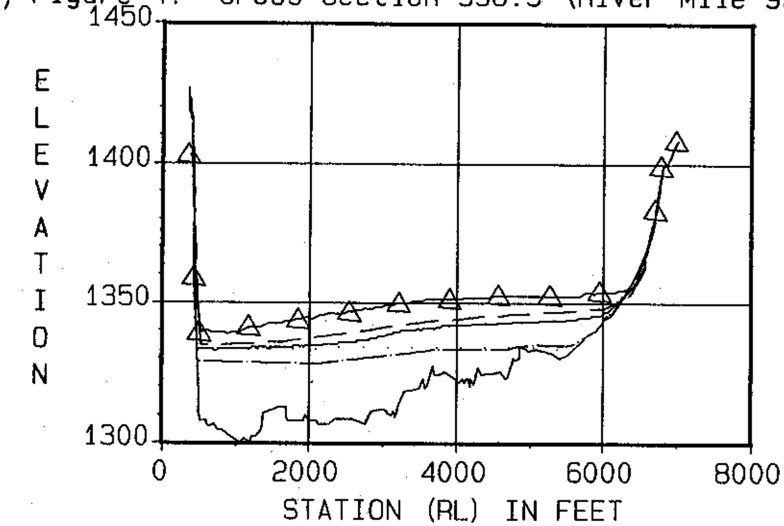


Figure 4. Cross Section 996.9 (River Mile 951.73)



——— 1953 —·— 1967 — — — 1977

······ 1962 ——— 1973 —△— 1986

Figure 5. Cross Section 1021.7 (River Mile 975.77

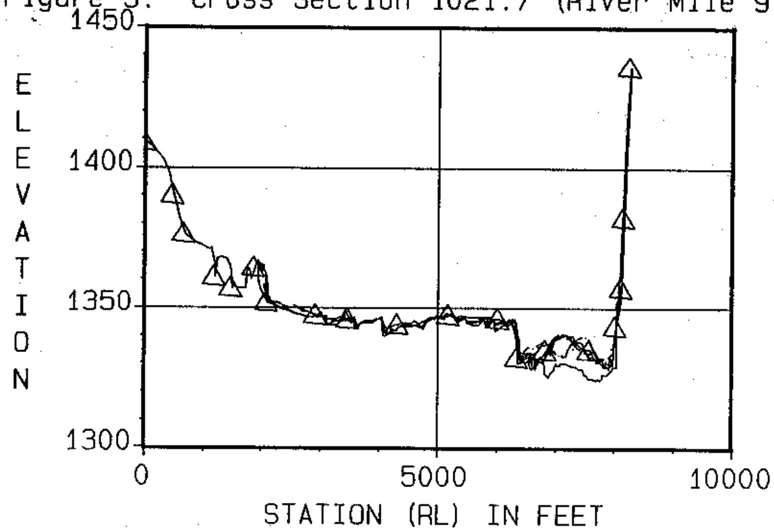
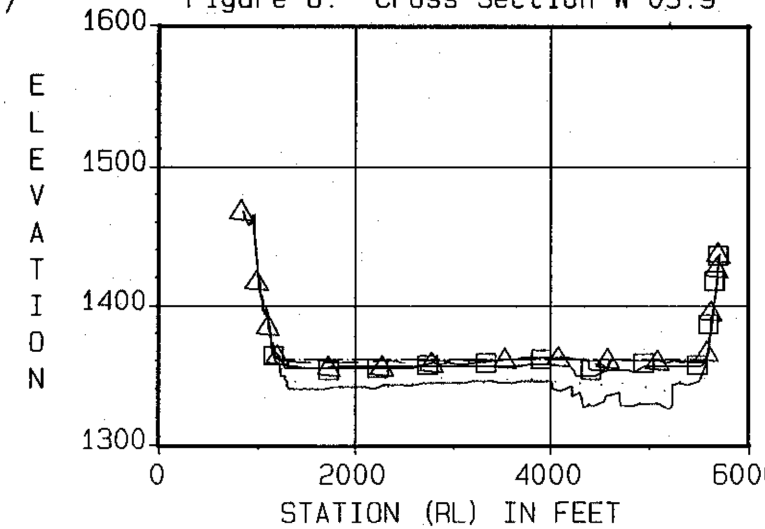


Figure 6. Cross Section W-03.9



RESERVOIR CAPACITY AND DEPLETION

The primary process responsible for the depletion of reservoir capacity in Lake Francis Case is the delivery and deposition of river bourn sediments from the White and, to a lesser extent, the Missouri Rivers. Reservoir volumes and sediment depletion rates at various pool elevations for 1953 and 1986 are presented in Table 5. The gross available storage for 1953, excluding the area upstream from Big Bend Dam, was 6,208,500 acre-feet at the maximum operating pool elevation of 1375 feet MSL. By the 1986 survey, this quantity decreased to 5,493,900 acre-feet. This is a total reduction of 714,600 acre-feet or 11.5 percent of the original volume. The annual rate of depletion is 21,500 acre-feet for the 33.2 year period of record. At this rate of depletion, the estimated life for the entire reservoir capacity up to elevation 1,375 feet MSL is 287 years, ignoring compaction and consolidation of sediments. In a like manner, the sediment inactive pool volume of 1,953,000 acre-feet would last only 91 years if sediment deposition continues at the present rate. Since the project was 33 years old (in 1986), the inactive pool volume will be lost to sediment deposition by the year 2044. From that time onward, sediment deposition would start encroaching the multipurpose and flood control pool volume.

TABLE 5
RESERVOIR VOLUME AND SEDIMENT DEPLETION

Elevation (feet-MSL)	1953 Capacity (ac-feet)	1986 Capacity (ac-feet)	Sediment Depletion Rate (ac-feet/year)	Percent Capacity Lost
1375	6,208,534	5,493,936	21,500	11.5
1365	5,229,240	4,508,064	21,708	13.8
1340	3,156,164	2,504,173	19,626	20.7
1320	1,953,477	1,544,734	12,304	20.9

SHORELINE EROSION

Shoreline erosion at Lake Francis Case is the result of wave action. Wave energy impacting on the shoreline displaces the in-situ erodible bank material to initiate the erosion process. The primary parameters which govern the incidence of waves on the reservoir are the wind speed, direction and duration; the fetch length; and the average water depth. Bank material at the lake is composed of highly weathered shales with occasional glacial tills and deposits.

Significant bank erosion can be observed for both banks of Lake Francis Case for most of the reach downstream of the White River Confluence. In this reach lie the wider sections of the reservoir, and the areas with the longest straight segments. The retreat face on the eroding banks typically corresponds to the historic maximum pool elevation -- 1365 feet MSL, down to the aggrading bed level. Shoreline retreat in this vicinity since 1953 is generally in the range of 30 to 150 feet, with a maximum of 200 feet. These values correspond to average annual rates of about 1, 5, and 6 feet respectively. Overall for the entire reservoir, the data indicate an average retreat of about 65 feet between 1953 and 1986, or about 2 feet per year. The north and east banks of the lake appear to be slightly more vulnerable to erosion than the south and west banks in the lower Missouri River reach.

Upstream from the White River confluence, erosion and shoreline retreat is comparatively moderate. The maximum retreat for the period of record is about 120 feet but more typical values are in the range of 10 to 50 feet -- a probable reflection of the narrower lake width in this reach. Similarly, retreat rates for the White River Arm are relatively minor (less than 20 feet since 1953). Again, for the White River, the lake width is narrow. Additionally, sediment bed elevations are above 1360 feet MSL which means that the water surface (and waves) only reach the banks at the highest pool elevations and the shallower water means smaller waves. Shoreline erosion for the entire reservoir is displayed graphically (note the exaggerated scale for erosion) in Figure 7.

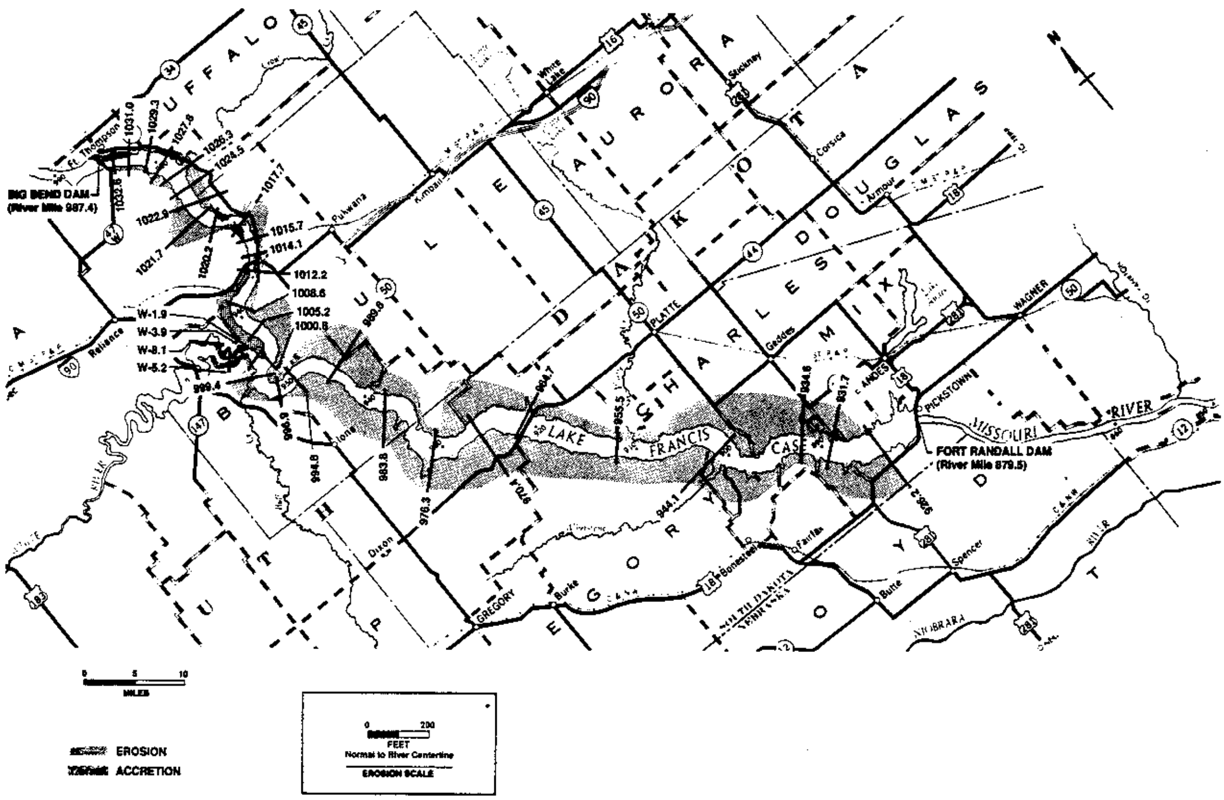


Figure 7. Shoreline Erosion 1953 to 1986

CONCLUSION

A brief overview of historic sedimentation data and trends for Lake Francis Case and the White River has been presented. Sedimentation in the lake has reduced its original capacity from 6,209,000 acre-feet in 1953 to 5,494,000 acre-feet in 1986 -- a reduction of 11.5 percent. Sediment deposition occurs throughout Lake Francis Case, with the heaviest aggradation occurring in the White River arm and in the reach 25 miles downstream from the White River confluence. This delta formation reflects the White River as being the primary current source of sediment inflow. Approximately 320 acres of perimeter land surrounding the reservoir have been lost to bank erosion, with bank retreats of up to 200 feet measured in the 33 year period of record up to 1986. The average retreat along the main banks of the reservoir is about 65 feet (2 feet per year).

SEDIMENTATION RATES AND CAPACITY RESTORATION OPTIONS, LAGO LOIZA, PUERTO RICO

by Paul D. Collar and Senén Guzmán-Ríos U.S. Geological Survey, San Juan, PR

ABSTRACT

The capacity of Lago Loíza, which supplies half the potable water for the metropolitan San Juan area, has been reduced about 70 percent by sedimentation during its 38 years of operation. Analysis of past bathymetric surveys indicates the reservoir could be completely filled with sediment as early as 1998. Therefore, sediment-mitigation strategies are needed if the long-term utility of Lago Loíza as an adequate water supply is to be maintained. The existing reservoir capacity is insufficient to meet current water demands during droughts similar to those of 1968 and 1974. Therefore, to ensure a constant water supply, a strategy is needed that will increase reservoir capacity and not one that will merely reduce the rate of sedimentation. This paper presents the calculations used to predict the useful life of the reservoir and presents a brief discussion of the feasibility of the sediment-mitigation alternatives practices being considered by the Puerto Rico Aqueduct and Sewer Authority. These alternatives include: (1) storm-water flushing over the spillway, (2) storm-water flushing through low-level outlets, (3) storm-water flushing with explosive resuspension of bottom material, (4) bottom-material siphoning, and (5) dredging.

THE STUDY AREA

The Río Grande de Loíza basin is located in northeastern Puerto Rico (see Gellis & Figueroa, this volume, figure 1 for map of study area) and is drained by Río Grande de Loíza, Río Gurabo, and numerous smaller tributaries. The drainage basin above Lago Loíza has a total area of 209 square miles. The geology of the study area was mapped by Briggs and Akers (1965). Intrusive granodiorites, diorites, and tonalites associated with the Cretaceous San Lorenzo batholith are exposed in the southern part of the basin. Cretaceous volcaniclastic tuffs, breccias, and associated rocks form the bedrock lithology in the rest of the watershed. Alluvium has been deposited in floodplains of the larger streams.

RESERVOIR CHARACTERISTICS

The Río Grande de Loíza was impounded in 1954 by a gravity dam with tainter gates (fig. 1) to form Lago Loíza, which had an original capacity of 21,760 acre-ft (acre-feet). Three bottom sluice-gates were built into the dam for sediment venting, and three penstocks were included for power generation. Subsequent to dam construction, concrete collars were poured around the valve stems of the sluices, rendering them inoperative. Hydroelectric power generation proved uneconomic due to the infrequency of reservoir releases; the gen-

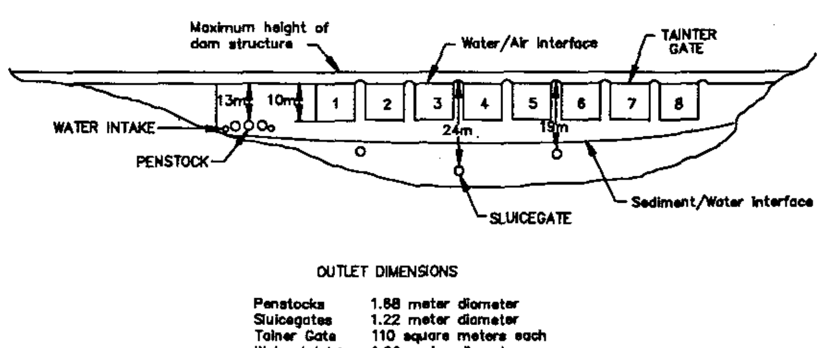


Figure 1. Loíza dam characteristics (modified after Puerto Rico Water Resources Authority, 1978).

Table 1. <u>Summary of all bathymetric surveys of Lago Loíza. Intersurvey and overall sedimentation rates have been divided by a normalization factor derived from figure 3.</u>

YEAR	REFERENCE	CAPACITY (acre-ft)	TOTAL SEDIMENT ACCUMULATION (acre-ft)	OVERALL SEDIMENTATION RATE (acre-ft/yr)	INTER-SURVEY SEDIMENTATION RATE (acre-ft/yr)	INTER-SURVEY SEDIMENTATION RATE (1974 OMITTED) (acre-ft/yr)
1954	Iivary, 1981	21,760	0	--	--	--
1963	Guzmán, 1963	18,707	3,053	339	339	339
1971	Hunt, 1975	15,628	6,132	361	385	385
1974	Quiñones, 1980	12,100	9,660	483	1176	--
1979	Iivary, 1981	14,771	9,394	376	-53	408
1985	Quiñones and others, 1985	10,100	14,065	454	779	779

erators have been removed, but the turbines are still in place. In 1977, flashboards were added to the dam to raise the maximum pool elevation 1 m (meter), thereby increasing the reservoir capacity by 2,400 acre-ft.

Five bathymetric surveys have been conducted in Lago Loíza since impoundment (table 1). The Soil Conservation Service (SCS) used 26 bathymetric profiles to estimate reservoir capacity using the range method (U.S. Department of Agriculture, Soil Conservation Service, 1983) in the 1963, 1971, and 1979 surveys. The U.S. Geological Survey (USGS) used 56 profiles in 1974 and 63 profiles in 1985 to estimate capacity with the range/line method.

SEDIMENTATION RATE OF LAGO LOIZA

Sedimentation rates for Lago Loíza were calculated based on the capacities determined from past bathymetric surveys. One implicit assumption in these calculations is that the different surveys are intercomparable. This may not be completely valid due to the fact that different numbers of ranges were used in the SCS and USGS studies.

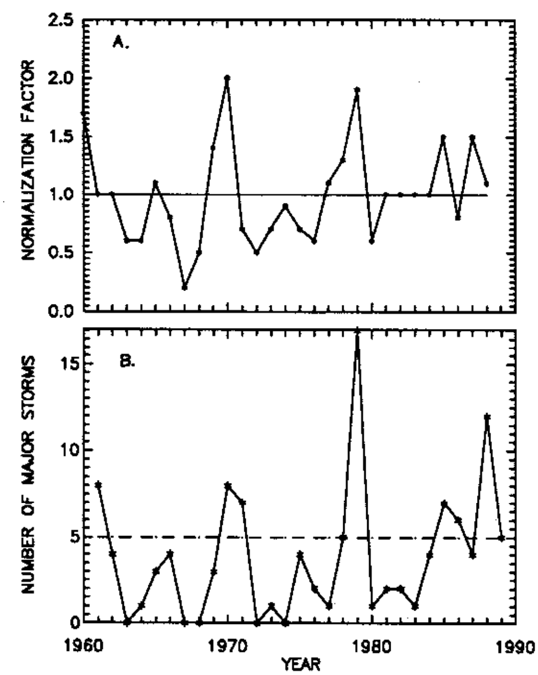


Figure 2. A. Plot of normalization factor against time. The horizontal line represents the long-term combined mean discharge. B. Number of storms causing a combined mean daily discharge one order of magnitude or more greater than the long-term combined mean daily discharge. The dashed line represents the average number of such storms per year.

For each of the capacity calculations shown in table 1, sedimentation rates were determined for the period since impoundment (overall sedimentation rate) and for the period since the previous survey (intersurvey sedimentation rate). Raw sedimentation rates were normalized with respect to average yearly reservoir inflow (fig. 2). This was done in order to dampen the response of sedimentation rates to discharge fluctuations and to amplify the long-term trends in sedimentation rate. The background sedimentation trends evaluated in this manner are thought to better reflect the effects of changing land-use patterns on sediment yields in the upper Loíza watershed than are sedimentation rates uncorrected for discharge.

Normalization involved calculation of mean daily discharges for each year of the period of record (1959-1989) divided by the long-term mean daily discharge. This quantity is plotted in figure 2 and

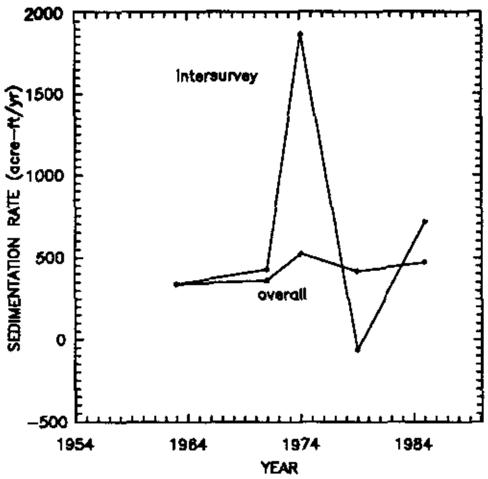


Figure 3. Plot of discharge-normalized overall and intersurvey sedimentation rates against years.

provides an index of the severity of wet and dry periods. Sedimentation rates were determined by dividing the volume of sediment accumulation by the number of years in the period for which the rate was being calculated. This sedimentation rate was in turn divided by the average normalization factor during the same period. The technique is illustrated with a calculation of the discharge-normalized sedimentation rate for the period 1979-1985. From table 1, the sediment accumulation during this period was 4,671 acre-ft. The raw sedimentation rate during this 6 year period was 779 acre-ft/yr (acre-feet/year). From figure 2, the average normalization factor during this period was (1.9 + 0.6 + 1.0 + 1.0 + 1.0 + 1.0)/6, or 1.1. Consequently, the discharge-normalized intersurvey sedimentation rate during the period 1979-85 was 779/1.1, or 708 acre-ft/yr.

The anomalously high overall and intersurvey sedimentation rates calculated for the period 1971-1974 (fig. 3) coincided with a period of drought in the Loíza basin (fig. 2), and the rates estimated for this period are correspondingly suspect. Fluvial transport of sediment has been shown to occur predominantly during high flows associated with large storms. The number of storms that resulted in a mean daily inflow to Lago Loíza greater than or equal to 3,700 ft^3/s (cubic feet per second) (10 times the combined long-term mean daily discharges of Río Grande de Loíza and Río Gurabo) is shown in figure 2. This graph shows that the period 1971-1974 was not only unusually dry but also had fewer major storms than average. The observations presented were used to justify the exclusion of the 1974 sedimentation survey from the evaluation of long-term sedimentation rates.

Excluding data from 1974, both overall and intersurvey discharge-normalized sedimentation rates increased as a function of time (fig. 4). Exponential regressions provided the best mathematical fits to the two trends. The empirical relations graphed in figure 4 were evaluated for a period of 54 years, and the calculated sediment accumulations were summed to project the relation between volume of accumulated sediment and time into the future. The accumulation of sediment predicted by the overall and intersurvey sedimentation rate

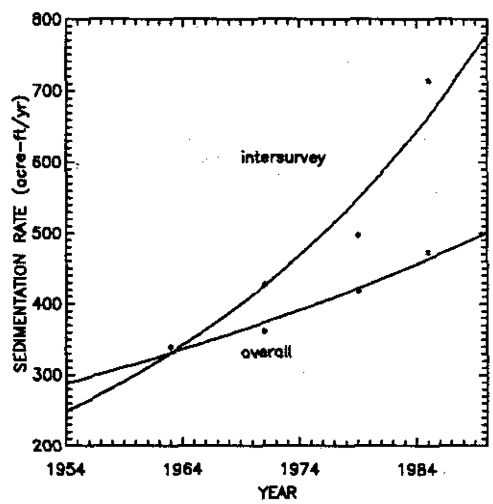


Figure 4. Plot of overall and intersurvey discharge-normalized sedimentation rates against years, excluding calculations based on the 1974 capacity estimate. The exponential regressions yielded $R^2_{overall} = 0.96$ and $R^2_{intersurvey} = 0.94$.

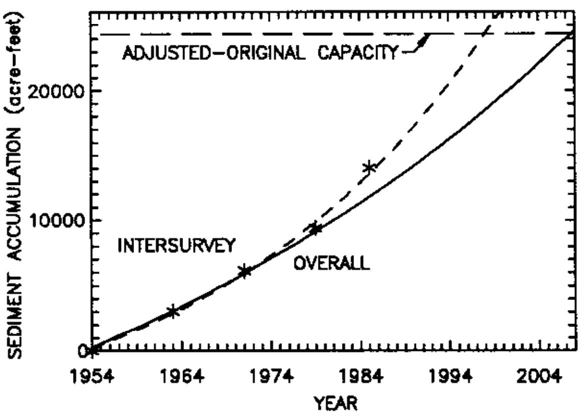


Figure 5. Plot of sediment accumulation predicted by overall (solid) and intersurvey (dashed) sedimentation rates against time. Asterisks represent actual capacity measurements.

regressions and the actual, measured sediment accumulations are shown in figure 5. The accumulation predicted by the intersurvey sedimentation rate provides a better fit to the actual sediment accumulation than does overall sedimentation rate. The overall sedimentation rate appears to be damped by the long period it represents and appears less sensitive to recent changes in sedimentation than is the intersurvey sedimentation rate. The accumulation predicted by the intersurvey sedimentation rate indicates that the capacity of Loíza reservoir will be exhausted as early as the year 1998. The exponential increase in Lago Loíza sedimentation rate is thought to reflect increased erosion resulting from urbanization in the Caguas area and the concomitant proliferation of housing development projects (A.E. Gellis and C. Figueroa, Hydrologists, U.S. Geological Survey, San Juan, PR; written commun., 1990).

The total weight of suspended sediment carried by the ríos Grande de Loíza and Gurabo during the period 1985-1989 (Guzmán-Ríos, 1990) was calculated to be 3.8 million tons (A. Gellis; oral commun., 1990). Using the specific weight of Lago Loíza sediment of 47.7 pounds per cubic foot reported by Iivary (1981), this corresponds to a volume of 3,660 acre-feet. To estimate the rate of reservoir sedimentation from 1985-1989 based on this sediment inflow, corrections were applied to compensate for the ungaged area discharging sediment to the reservoir (28 percent), trap efficiency, and the estimated proportion of inflow occurring as bedload. The variation of trap efficiency through time was calculated using the long-term average yearly inflow of 337,370 acre-ft (28-year mean yearly discharges of ríos Gurabo and Grande de Loíza plus 28 percent to account for the ungaged area) and is shown on Brune's (1953) curve in figure 6. From figure 6 an average trap efficiency of 65 percent was determined for the period 1985-1990. Bedload was calculated to be 14 percent of the total sediment inflow with the Shoklitsch Formula (A. Gellis; oral commun., 1990). Total-sediment ac-

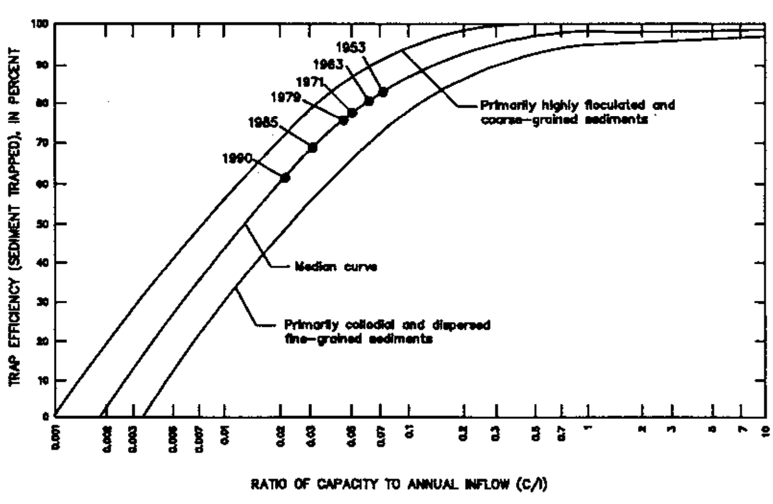


Figure 6. Brune's (1953) curve with plots of Lago Loíza data corresponding to the years 1953, 1963, 1971, 1979, 1985 (based on capacity measurements determined by bathymetric surveys), and 1990 (based on capacity predicted by the intersurvey sedimentation rate function given in figure 5).

cumulation during the period 1985 to 1990 was calculated as follows: $\{[(3.8 \times 10^6 \text{ tons} \times 2{,}000 \text{ lbs/ton}) / 47.7 \text{ lbs/ft}^3] / 43{,}560 \text{ ft}^3\text{/acre-ft}\} \times 1.28 \times 1.14 \times 0.65 = 3{,}470$ acre-ft. Adding the sediment accumulated between 1985 and 1990 to the 1985 measured sediment accumulation of 14,070 acre-ft yields a 1989 estimated sediment accumulation of 17,540 acre-ft. The empirical relation between intersurvey sedimentation rate and time shown in figure 4 predicts a sediment accumulation of 16,441 acre-ft in 1989 (fig. 5). The difference between these independent estimates is six percent.

SEDIMENT MITIGATION ALTERNATIVES

The three primary constraints governing the selection of an optimal sediment-mitigation strategy are: 1) reduction in quality and quantity of municipal water supply, 2) downstream environmental degradation, and 3) cost. Sediment mitigation alternatives which may prove useful in partially restoring lost capacity or minimizing future capacity losses that are being considered by the Puerto Rico Aqueduct and Sewer Authority include: 1) storm-water flushing over spillway 2) storm-water flushing through low-level outlets and over spillway, 3) storm-water flushing combined with explosive resuspension of bottom material, 4) bottom-material siphoning, and 5) dredging. These alternatives are discussed in the following sections.

Storm-Water Flushing Over Spillway

Storm-water flushing takes advantage of the fact that sediment transport occurs predominantly during large storms. Lowering of the reservoir pool elevation before a major storm decreases the cross-sectional area of the reservoir and increases the velocity of the storm runoff flowing through the reservoir. This enables particles with relatively high settling velocities, which at design pool elevation would be trapped by the dam, to be transported through the reservoir. Storm-water flushing slows the rate of sedimentation but does not restore capacity already lost to sedimentation.

A storm-water flushing strategy is currently being implemented by the Puerto Rico Aqueduct and Sewer Authority (Gregory Morris, Gregory Morris and Associates, oral commun., 1990). A real-time rainfall/runoff model currently being calibrated to the Loíza basin is expected to provide the capability to predict major runoff events with a lead time of 2 to 3 hours (Heriberto Torres, U.S. Geological Survey, oral commun., 1990). This will enable optimal timing of gate openings and closures. Flushing over the spillway is a relatively inexpensive alternative, because the correct timing of gate openings and closures is the principal variable controlling implementation. Storm-water flushing over the spillway would not require cessation of water production, because intakes are located 3 meters beneath the spillway elevation. This alternative would probably also have the least environmental impact of the alternatives considered, because the oxidized, surficial sediment in transport is chemically stable and is likely to be transported all the way to the ocean by the same storm event.

An analysis of flow records from the Loíza basin during the drought periods 1966-1968 and 1971-1974 was conducted to determine how quickly the projected 1990 capacity of 7,765 acre-ft (2,530 million gallons) would be depleted by the current water-supply demand of 78 Mg/d. Combined mean monthly river inflows to Lago Loíza were 31, 25, and 35 Mg/d during February, March, and April, 1968. Production of 78 Mg/d would require continuous withdrawals from storage, which would deplete the 1990 storage capacity (from figure 6) in 53 days. This estimate neglects the inflows from minor tributaries as well as additional water losses from evapotranspiration. A recurrent drought equivalent in severity to that experienced in May, June, and July, 1974, would deplete the reservoir capacity in 75 days. The recurrence intervals of these drought periods are from 15 to 26 and 10 to 27 years, respectively (Colón-Dieppa and others, written commun., 1990). Consequently, a sediment-mitigation program which does not remove accumulated sediment will not provide the capacity needed for long-term withdrawals at present levels during periods of major drought.

Storm-Water Flushing Through Low-Level Outlets

An evaluation of worldwide reservoir sediment-mitigation procedures (UNESCO, 1985) found that flushing of sediments through low-level outlets was the most effective overall means of partially restoring lost capacity. Low-level outlets are effective because increased water velocities caused by the outlets promote sediment scour and because sluice-gates can be used for venting of density currents.

Implementation of a flushing strategy designed to increase the capacity of Lago Loíza will require the restoration of the sluice-gates, or the construction of new ones. If the sluice-gates in place can be rendered operational, some dredging may be necessary to remove the sediment accumulated above the sluice-gates inside the reservoir. If the existing sluice-gates cannot be returned to service, installation of additional sluice-gates would be costly. Penstocks used as outlet structures would provide an additional 3 m of static advantage over use of the spillway for flushing, but their location to the far eastern side of the dam (fig. 2) may minimize their effectiveness in flushing sediment through the reservoir. Furthermore, the penstocks may not be low enough to promote overall scour during flushing events.

The discharge of scoured bottom material through sluice-gates could threaten the water quality of the downstream reach and of the coastal waters near the mouth of the Río Grande de Loíza. Whole sediment concentrations of major, minor, and trace elements determined from the analysis of 56 bottom-sediment cores collected in Lago Loíza in 1989 are given in table 2. Although toxic metals were not found in dangerously high concentrations, the mean concentrations of cobalt, copper, zinc, chromium, nickel, iron, and mercury were higher than average lake sediment concentrations in the United States (Horowitz and others, 1989). The decomposition of organic material in the bottom sediment has created a highly reducing chemical environment, evidenced by outgassing of methane. Because the solubility of most metals increases in reducing environments, sediment pore water could contain relatively high concentrations of some metals. The scouring of sediment would

Table 2. Whole sediment trace metal concentration ranges and means calculated from analyses of 56 cores of the upper four feet of lago Loíza sediment. Lake sediment means are extracted from Horowitz and others (1989) [mg/kg, milligrams per kilogram].

TRACE METAL	LOIZA MINI-MUM (mg/kg)	LOIZA MAXI-MUM (mg/kg)	LOIZA MEAN (mg/kg)	U.S. LAKE SEDIMENT MEAN (mg/kg)	METAL CONCENTRATION IN LAGO LOIZA EXCEEDS U.S. LAKE SEDIMENT MEAN
Arsenic	1.8	5.9	3.5	7.1	no
Selenium	0.3	1.5	0.6		
Barium	349	1070	490		
Cerium	15	30	24		
Cobalt	19	32	26	15	yes
Chromium	56	131	70	36	yes
Copper	68	131	105	13	yes
Gallium	19	26	22		
Lanthanum	8	18	13		
Lithium	10	21	15		
Neodysium	8	22	15		
Nickel	21	207	31	19	yes
Lead	10	27	18	21	no
Scandium	8	32	28		
Strontium	97	265	173		
Vanadium	152	271	198		
Yttrium	10	28	21		
Ytterbium	1	4	2.8		
Zinc	84	151	132	62	yes
Mercury	0.12	0.38	0.21	0.05	yes

result in the mixing of potentially metalliferous interstitial water and storm water. Mixing would probably result in the mobilization of metals, which would increase their bio-availability. A simultaneous downstream water threat would result from the resuspension of organic material, which would probably exert an oxygen demand on the downstream reach. However, the volume of sediment and interstitial water scoured during a storm will probably be small compared to the volume of the stormflow. Except for suspended-sediment loads, water-quality degradation in the downriver reach is likely to be mitigated somewhat by the dilution effect and the rapid travel time of river water moving from the dam to the mouth of the river.

Storm-Water Flushing with Explosive Resuspension of Bottom Material

The detonation of low seismic velocity explosives in bottom sediment timed to coincide with the passage of the discharge peak through the reservoir may result in the resuspension of bottom material and artificially trigger density currents. If bottom outlets can be restored, explosive resuspension may complement storm-water flushing in two ways.

1. The development of a deep narrow central channel in the deposited sediments is a problem that frequently accompanies flushing through a restricted number of low-level outlets. Directional blasting on both sides of the central channel may promote underwater slumping of the channel walls and result in density-current transport of mobilized sediment along the old channel bed.

2. The detonation of explosives along the more steeply sloping portions of the delta face may promote underwater mass-wasting of the silt and sand and similarly cause turbidity-current transport of sediments, which could be vented through sluice-gates.

Because a greater volume of sediment would be potentially released through the use of explosives than through low-level storm flushing alone, the environmental impact of explosive resuspension would probably be greater than that associated with previous alternatives. A pilot-testing program of this strategy would be needed to assess costs and benefits, to model downstream depositional patterns, and to determine the optimal type, quantity, and arrangement of explosives.

Bottom-Material Siphoning

Siphoning bottom sediments has been used to successfully restore capacity in reservoirs (Fan, 1985). The technique consists of routing a large diameter flexible hose over or through the dam. In Lago Loíza, the existing penstocks could be used to pass the hose through the dam. One end is allowed to move freely within an area inside the reservoir. The other end of the hose is placed at the toe of the dam. Once the siphon is activated by priming, continual discharge is accomplished as a result of the difference in head between the inlet and outlet of the hose. Increased sediment discharges can be encouraged by installation of a hydraulically driven cutter head on the intake end of the siphon. Alternately, small explosive charges might be employed to loosen sediment immediately prior to siphoning. This mitigation alternative does not depend upon storm flow and could be operated routinely during the wet season in Lago Loíza. Because of low downstream flows when the reservoir is not releasing water, however, the high-sediment discharge would not be diluted as much as for previously discussed alternatives. The discharge of siphoned material could consequently result in an excessive downstream oxygen demand and the potential mobilization of environmentally active trace metals. For bottom-material siphoning at conditions other than high flows, downstream sediment deposition would be expected during routine operation, although deposited sediment would probably be resuspended and transported to the ocean during periodic large storms. If bottom siphons were operated prior to and during reservoir releases, effects on downstream water quality would be reduced, but the volume of removed material might not warrant the costs of implementation.

Bottom siphoning has been used in France to remove sediment from around the water-supply intakes of Ríoumajou Reservoir (Evrard, 1980). The simultaneous operation of intakes and siphon resulted in a decrease in the suspended-sediment concentration of intake water and a concomitant decrease in water treatment costs.

Dredging

Although the feasibility of dredging bottom materials from Lago Loíza has never been formally studied, this alternative has two significant drawbacks. First, dredging is time-consuming, labor-intensive, and costly. Second, convenient sites for disposal of sediment have not been identified. If spoils were transported offsite for application as topsoil conditioner, used in land-reclamation projects, or disposed of within landfills, transportation costs and infrastructural limitations are likely to be substantial. If dredged spoils can be introduced into the Río Grande de Loíza below the dam, preliminary evaluations of stream hydraulics indicate that high flows have sufficient energy to transport silts and clays past the mouth of the river and into the Atlantic Ocean.

The impacts of dredging on potable water production depend on the technology employed. Operation of a drag-line would probably require the cessation of water production as a result of the high turbidity caused by sediment resuspension. Operation of dredges with pumps mounted at the surface would require substantial lowering of the reservoir pool elevation, which may also require halting water production. The use of a dredge equipped with a pump mounted on the cutter-head may enable the uninterrupted production of low-turbidity water simultaneous with dredging activities.

The economics of the raw-materials market in Puerto Rico may facilitate cost-effective dredging of upstream parts of the reservoir. The wholesale price of sand in Puerto Rico is currently about $18.00 per cubic meter. Cores of the upper 4 feet of sediment in the reservoir had sand fractions ranging from 80 percent at the delta to 4 percent near the dam; the sand content averages 22 percent. At current market value, there is about $63 million dollars worth of sand in the upper four feet of bottom sediment. In the absence of deep coring data, it is unknown as to whether the sediment size distribution determined for the upper four feet was representative of the deposit as a whole. The reclaiming of sand dredged from the reservoir bottom could pay for part of the costs of dredging and disposal of the fines.

Environmental impacts associated with dredging include the disposal of the dredged material and the effluent stream. Elutriate tests will be needed to determine the chemical stability of the solids, which will indicate acceptable disposal alternatives, including the feasibility of flushing clays and silts downstream during large storm events.

CONCLUSIONS

Barring the successful implementation of sediment mitigation efforts, the capacity of Lago Loíza could be exhausted as early as 1998. The existing capacity of the reservoir is insufficient to sustain current water demands through drought periods as severe as those of either 1968 or 1974. The implementation of a flushing strategy which does not remove previously deposited sediment will result in water shortfalls during prolonged dry periods. Implementation of a flushing strategy designed to partially restore lost capacity will require the restoration of low-level outlets. Low-level flushing might be enhanced through the detonation of explosives in the bottom sediments simultaneous with the passage of storm peaks through the reservoir. Explosive resuspension of sediment would not significantly enhance storm-water flushing if the spillway is the only outlet structure utilized. Bottom-material siphoning is probably not a viable alternative due to downstream water-quality impacts except during wet periods when water is regularly discharged from the reservoir. The expense of dredging could be offset by focusing sediment removal efforts in the delta and by reclaiming sand from the dredged material. The optimal selection of mitigation alternatives is likely to involve a combination of more than one of the possibilities discussed.

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PHOSPHORUS TRANSPORT AND WATERSHED SCALE

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ABSTRACT

The scale dependence of the relation between suspended sediment and phosphorus (P) is investigated for the Washita Basin, Oklahoma. Sediment, particulate P (PP), and soluble P (SP) were measured in watersheds ranging in size from 0.6 to 1,865,000 ha. A continuous PP-sediment relation appears to hold within and across all watershed scales. The uniformity of the relation is attributed to similar transport processes within each scale, and to the proportionality of the size of upstream sediment and P sources and instream transport capacity with watershed scale. This suggests that sediment and P yield at large watershed scales can be estimated from an adequate coverage of representative source area watersheds to encompass spatial and temporal variations in climate, soils, geology, and land use. More subtle changes are revealed when sediment and P are expressed as relative quantities. For a given sediment concentration higher PP concentrations are observed at the source areas than at the larger watershed scales. This is attributed to the erosion of P-deficient sediment from stream banks. Further, a decrease in mean SP concentration with increasing scale is attributed to SP sorption by P-deficient eroded stream bank material, and to the dilution of stream flow by subsurface return flow of low SP concentration. Spatial variability of large natural areas and temporal variability of precipitation overshadow the subtle adjustments of the PP-sediment relation in downstream direction. This and limited data availability suggests physically-based numerical modeling as an alternative to quantify the details of P transport across watershed scales.

INTRODUCTION

Agricultural nonpoint-source pollution of surface waters via erosion and chemical transport in runoff is one of the major water quality concerns facing the U.S. (USEPA, 1984). The transport of phosphorus (P) can result in accelerated eutrophication of waterways and lakes, limiting their fishery, recreational, and industrial use. To evaluate these potential economic and environmental impacts of agriculture, the form and amount of P entering a downstream lake must be known. Phosphorus is transported in soluble (SP) and particulate (PP) forms. Soluble P is immediately available for algal uptake. Particulate P includes inorganic P sorbed by soil particles and organic matter eroded during runoff and may provide a long-term source of P for aquatic growth. Particulate P constitutes the major proportion of P transported in runoff from cultivated land as P is strongly sorbed by clay-sized material and organic matter contains relatively high levels of P (Sharpley and Smith, 1990). Changes in sediment, PP, and SP occur during stream flow transport from source area to watershed outlet (Kunishi et al., 1972; Gburek and Heald, 1974; Sharpley and Syers, 1979). The direction and extent of these changes depend on the relative concentration of sediment, PP, and SP, sediment size distribution, sediment source (suspended, stream bank, and bed material), and particle size sorting during transport. These changes must be taken into consideration when assessing the economic and environmental impacts of agricultural land use on downstream surface waters.

Most studies of land use and P transport in runoff have been conducted on small plots and at field scales (0.1 to 10 ha) with limited climatic, soil, topographic, and management variablity (Ryden et al., 1973; Johnson et al., 1976; Sharpley and Menzel, 1987). Further, limited information is available on P transport in runoff as a function of scale within a given watershed. Although several models have been developed to simulate sediment and P transport in runoff from agricultural land (DeCoursey, 1985), few attempts have been made to model this transport from small field (10 ha) to basin scales (10,000 to 50,000 ha) (Young et al., 1989). This paper presents an investigation of relations between sediment, PP, and SP transport as a function of watershed scale. The study is site specific to the Washita Basin, Little Washita River subwatersheds, and associated source areas in south central Oklahoma. Most of the data used in this study was collected between 1980 and 1985 by the USDA-ARS for other unrelated water quality investigations. Shortcomings of existing water quality data for scale investigations and further research needs are identified.

STUDY AREA AND METHODS

The location and characteristics of the source areas, Little Washita River subwatersheds, and Washita Basin are given in Figure 1 and Table 1. Average annual precipitation over the Washita Basin is about 550 mm on the western edge and 850 mm on the eastern edge. It is about 750 mm in the Little Washita Watershed where all the source areas are located. Most of the precipitation occurs in the spring and fall months. The Land Resource Areas in the Washita River Basin are primarily Western Rolling Red Plains, Cross Timbers and Central Reddish Prairie. The land use is, on the average, 25% cultivation, 65% range and 10% miscalleneous uses such as timber, rock, urban areas and reservoirs.

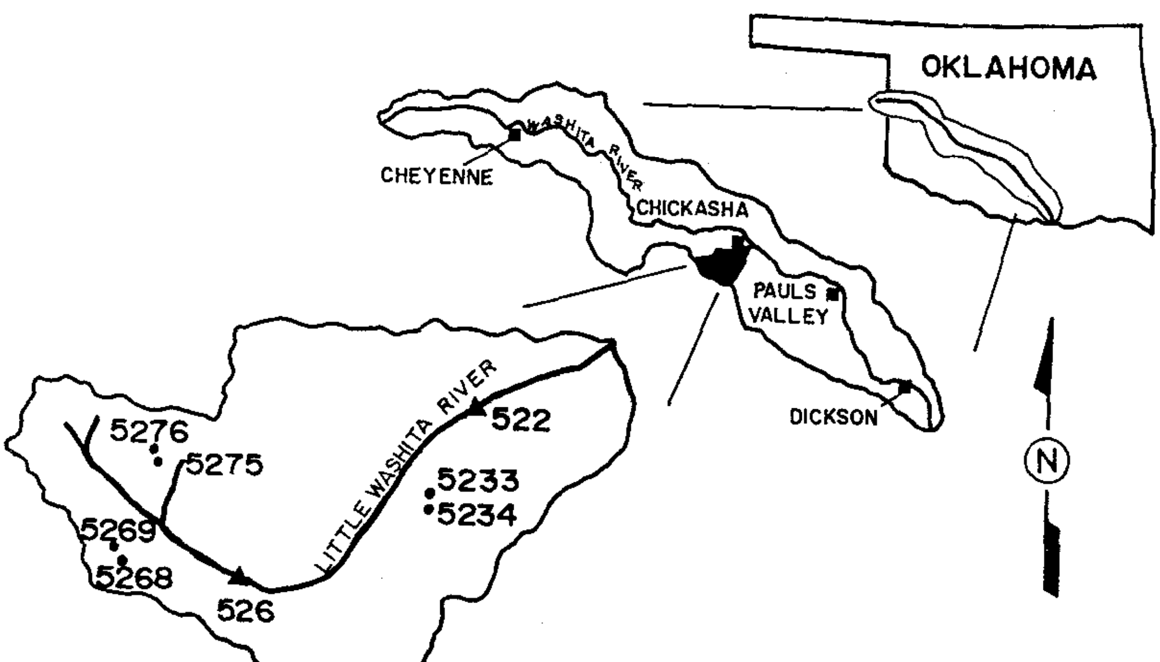


Fig. 1. Location of watersheds in Oklahoma, USA.

Table 1 Watershed characteristics.

Identif-ication	Size (ha)	Sampling Period	Number of samples	Landuse	Phosphorus application (kgP/ha/yr)
Source areas					
5233	2.3	1980-82	10	Grass	0
5234	1.2	1980-82	12	Grass	0
5268	3.1	1980-85	44	Grass	0
5269	4.2	1980-85	59	Grass	0
5275	0.6	1980-85	44	Crop	20
5276	0.6	1980-85	44	Crop	20
Little Washita subwatersheds					
526	16'025	1979-83	35	Mixed	-
522	53'794	1979-83	61	Mixed	-
Washita Basin					
3310	1'865'300	1980-87	47	Mixed	-

The smallest scale is represented by 4 grassed and 2 cultivated independent source areas (0.6 to 4.2 ha) in the Little Washita River Watershed. Although the number of source areas is adequate to establish trends in P loading at the small scale, they do not allow an accurate mass balance of sediment, PP, and SP loading between the various watershed scales. All runoff events during the study period were sampled, and suspended sediment, PP, and SP concentration determined. Suspended sediment (subsequently referred to as sediment) was determined as the difference in weight of 250 mL aliquots of unfiltered and filtered (0.45 um) samples, after evaporation to dryness. The concentration of SP and total P (TP) was determined on filtered (0.45 um) and unfiltered samples, respectively. Particulate P was calculated as the difference between SP and TP.

The intermediate scale is represented by 2 nested subwatersheds (16,025 ha and 53,794 ha) in the Little Washita River Watershed. The subwatersheds are defined by stream gages 526 and 522 on the Little Washita River (Fig. 1). The 6 source areas are nested within these two subwatersheds. Discharge and sediment data at gage numbers 526 and 522 are available on a daily basis. Particulate P and SP concentrations were determined periodically over a five year period. Only data for days having full water, sediment and water quality information are used in this study. The largest scale is represented by the Washita Basin (1,865,000 ha) above the USGS stream gage near Dickson, Oklahoma. Water quality data for this study were obtained from USGS Water-Data Reports (USGS, 1980-1987).

RESULTS

Particulate Phosphorus

A plot of PP versus sediment yield for all watershed scales is presented in Fig. 2. Sediment and PP yield are logarithmically related over all watershed scales. The positive relation is attributed to the increase in sediment and PP supply, as well as the increase in transport capacity of the hydrologic system with runoff and watershed scale. It is believed that PP yield at a given location depends primarily on upstream sediment erosion and yield and on local sediment transport characterisitics as a function of runoff. From one scale to another, i.e. in downstream direction, PP yield is believed to depend primarily on the spatially variable sediment supply, as well as on changes in sediment transport and size distribution in the downstream direction. As shown in Fig. 2 the same PP-sediment relation appears to hold within, as well as across, watershed scales. This suggests, as a first approximation, that PP yield on a basin scale, may be estimated from representative small scale watersheds. However, the variability of the data and the wide range of values produced by the strong dependence of the variables on drainage areas may mask more subtle, yet important scale effects that must be considered before small scale watershed data are extrapolated to a larger scale.

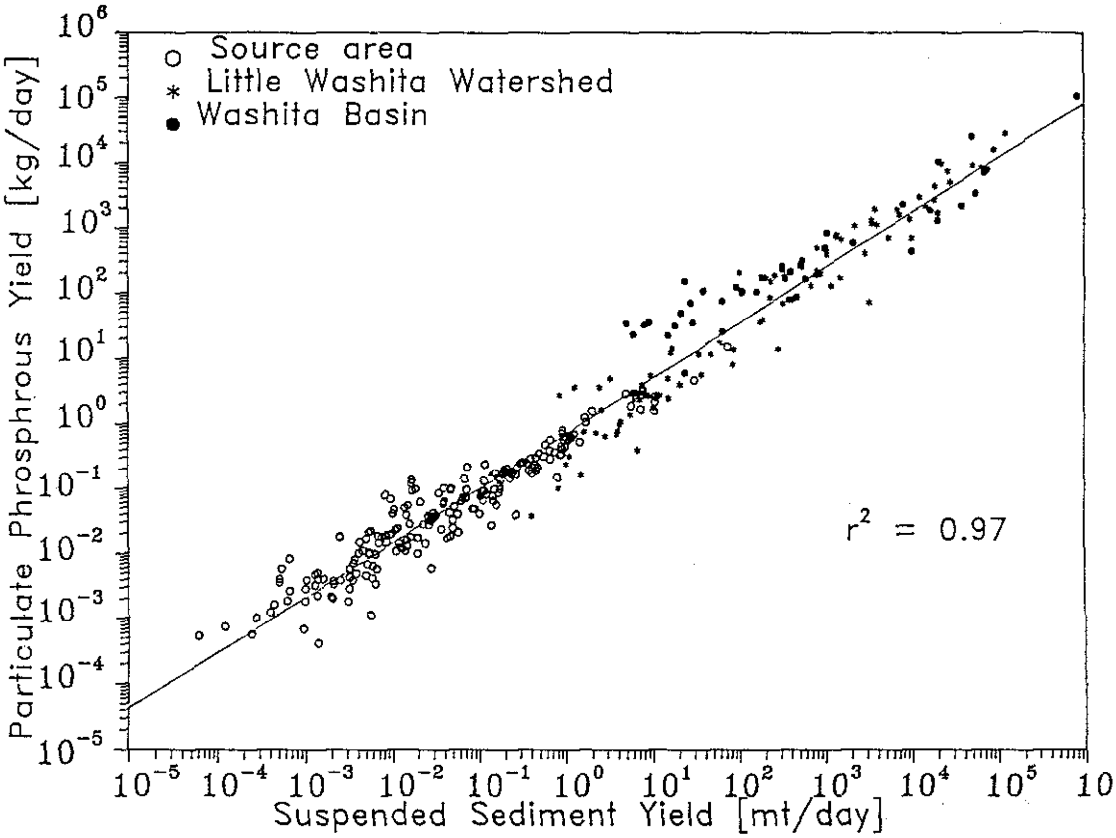


Fig. 2. Particulate P yield versus suspended sediment yield.

The direct effect of drainage area can be eliminated by expressing yield on a unit area or concentration basis. A logarithmic plot of PP versus sediment yield per unit area, produces a positive relation for each of the three scales as shown in Fig. 3. The similarity of the three relations suggests that same sediment and PP tranpsort processes as a function of runoff govern the PP-sediment yield relation within each scale. The slightly steeper slope of the relation for the Little Washita subwatersheds is attributed to the flood detention reservoirs, which trap sediment (Menzel et al., 1986). The sediment trapping is particularly effective at low flow conditions when outflow from the reservoirs are small and reservoir residence times are long. Also, the data in Figure 3 show a consistently lower PP yield for the larger, stream gaged, watersheds than on the source area watersheds. This shift is attributed to an increasing contribution of P-deficient sediment from stream bank erosion (Ryden et al, 1973).

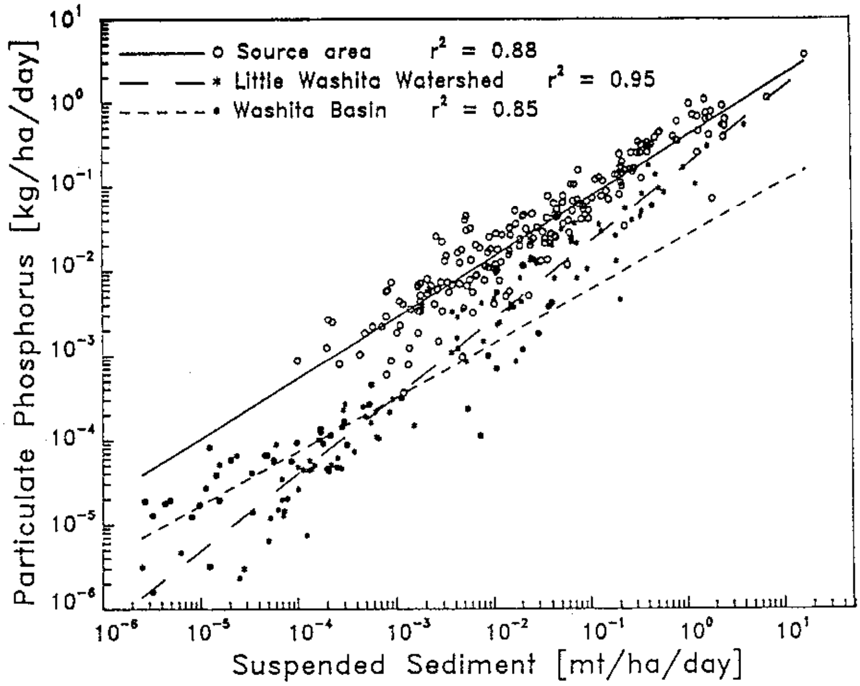
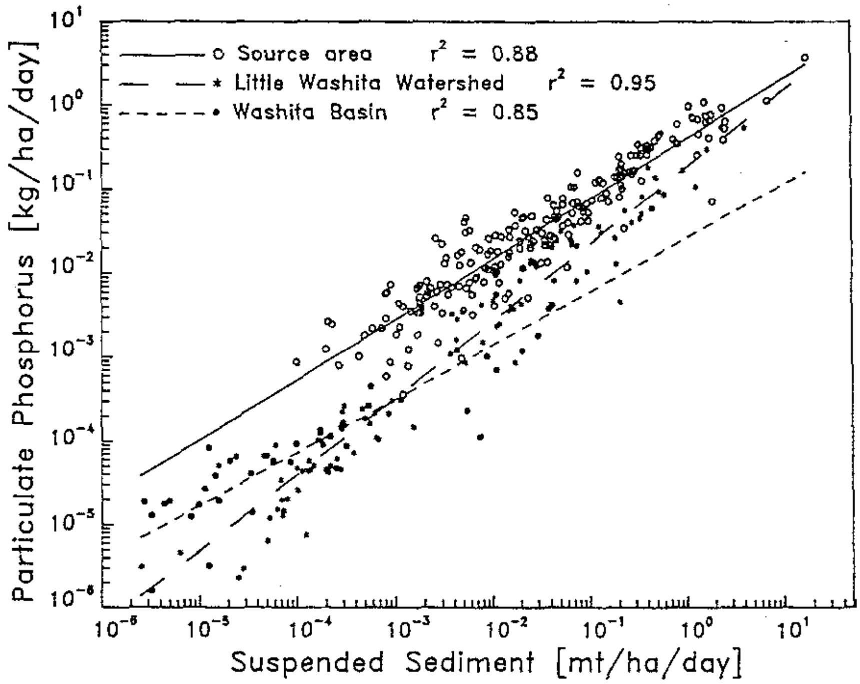


Fig. 3. Particulate P versus sediment yield on a unit area basis.

Figure 4 shows that PP content of runoff sediment decreases as sediment concentration increased for each of the three watershed scales. This is attributed to an increase in transport of silt-sized (>2 um) particles, of lower P content than finer clay-sized (<2 um) particles, as sediment concentration increases. Further, the effect is accentuated by an increasing contribution of P-deficient eroded stream bank and subsoil material to the transported sediment as stream flow and sediment concentration increased. This is evident from the steeper slope of the PP-sediment relation and the lower PP content at given sediment concentrations for the largest scale (Washita Basin). The smaller value of the slope of the relation for the Little Washita subwatersheds is again attributed to the sediment trapping effect of the flood retention reservoirs.

<u>Soluble Phosphorus</u>

While not presented here, a linear logarithmic relation is obtained between SP yield versus runoff volume. The same relation holds within, as well as across watershed scales for the same reasons previously stated for PP. No relation is observed between SP concentration versus runoff volume or sediment concentration. However, mean SP concentration decreases with increasing watershed scale as shown in Fig. 5. The mean SP concentration of runoff was higher from unfertilized grassed (0.14 ppm) than P-fertilized cropped watersheds (0.09 ppm), as a result of P leached from vegetative material in the former and sorption of SP by eroded soil in the latter land use. An approximate 50% reduction in mean SP concentration from the source areas to the largest watershed scale is observed. This scale effect is attributed to the sorption of SP by P-deficient sediment during stream flow (Sharpley et al., 1981), and to the diluting effect of low SP concentration subsurface return flow, the contribution of which increases with scale (Gburek and Heald, 1974).



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DISCUSSION

On a logarithmic plot PP yield increases linearly with sediment yield within and across watershed scales (Fig. 2), suggesting a scale independent relation. More subtle changes are observed when the variables are expressed on a unit area or concentration basis. Under these conditions the PP-sediment yield relations for each of the three watershed scales show shifted, yet almost parallel, trends (Fig. 3 and 4). The parallel trends suggest that similar fundamental transport processes govern the PP-sediment relations within each scale. The shift in the relation between watershed scales, particularly between source areas and the larger gaged watersheds, may be attributed to the influx of P-deficient sediment from bank erosion of concentrated flow. The resulting reduction in PP is offset to a certain degree by the sorption of SP, and by the effect of sediment size sorting in the downstream direction. Size sorting results in large particles of low P-content to be deposited, and fine particles of high P-content to be carried downstream. These counteracting effects, as well as the overriding effect of size and spatial variability of the supply area on runoff, sediment and PP must be taken into consideration when estimating PP yield across watershed scales.

Runoff, sediment and P varies greatly as a result of spatial and temporal variations in watershed characteristics and precipitation input. To obtain reliable relations long-term data are needed to overcome the natural variability of the environment, in particular for source areas, where a few large events can produce the major portion of the annual sediment and P yield. In addition to long-term data, corresponding compatible data at the different watershed scales must be available. Yet sampling agency, methods and parameters sampled, vary with scale. Also, the number of source areas needed for a representative mix of small watersheds is high because land use, geology, soils, and even climate may vary significantly within a large watershed. These data problems make an accurate analysis of sediment and P yield with scales very difficult. Trends may be evident, but quantitative analysis of interdependent cause-effect relations are at best speculative because field data are the integration of all upstream effects and do not readily lend themselves to disaggregation for individual processes.

The problem of data availability and interpretation can be alleviated by modeling the transport numerically. Sources and amounts of sediment and P, forms of P and any subsequent transformations during downstream transport can be quantified by numerical modelling at any location and at any time. Phosphorus yield at the field scale and as a function of land use and management, soil type, and weather can be described with existing numerical models. Additional modeling efforts are needed to quantify the cause-effect relations between the various P transport processes in the stream channels. Processes that must be considered in any future attempts to model P transport in streams and across watershed scales include: dilution by subsurface return flow; sediment supply from concentrated flow to account for P-deficient sediment; sorption/desorption of P with changing water and sediment characteristics in downstream direction; partitioning between SP and P;, PP enrichment in downstream direction; and last, but not least, a detailed mass accounting of water, sediment and P by source, and their mixing ratios at the different scales. Mass accounting by source and mixing ratios may provide the necessary insight to determine the location and type of agricultural management needed to achieve maximum improvement in downstream water quality.

CONCLUSIONS

With our present knowledge the effects of scale on the overall PP-sediment relation are primarily brought about by the strong dependence of the variables on drainage area. Similarities in sediment and PP transport with runoff at a location and in downstream direction produce a single relation within and across watershed scales. However, more subtle, yet important, scale effects are brought out by considering the variables on an unit area or concentration basis. Differences between scales, particularly between the source areas and the first watershed scale containing streams, are primarily attributed to the addition of P-deficient sediment from stream bank erosion, sorption of SP by this sediment, and dilution by subsurface return flow of low SP concentration. Spatial and temporal variability of watershed characteristics and precipitation input may override these subtle scale effects. Limitations of field data to disaggregate dependent cause-effect relations in P transport in streams and the limited availability of a consistent data set across scale support the use of physically-based numerical models to complement field data. The modeling of P transport in streams lends itself to compare land management alternatives and determine soil and water conservation measures having maximum potential improvement in downstream water quality.

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EFFECT OF CLIMATE FLUCTUATIONS ON WATERSHED RUNOFF

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ABSTRACT

Climate fluctuations, defined in this study in terms of annual precipitation, have a pronounced impact on watershed runoff and can easily mask beneficial effects of soil and water conservation measures. The effect of two major climate fluctuations, a dry (1963-1969) and wet (1981-1985) period, on water and sediment yield in the Little Washita River Watershed is presented. Water yield tripled and the frequency of floods over 500 cms quadrupled from the dry to the wet period. A previous study indicated that Flood Water Retardation Structures (FRS), which were installed in the 1970's and which controlled 45% of the drainage area, had little effect on water and sediment yield. The beneficial effect of the FRS, among other reasons, have been overshadowed by the climate fluctuations. Therefore, relevant hydrologic and meteorologic influences must be checked and, if necessary, adjustments made to avoid misinterpretations of the effects of soil and water conservation measures. The direct influence of climate fluctuations on sediment yield is, to some degree, avoided by interpreting sediment-discharge rating curves. Sediment yield was found to have decreased by about 50% presumably due to the construction of the FRS. Ultimately, a disaggregation of the effects of the FRS and those of climate fluctuations on watershed runoff may only be achieved with physically based numerical models.

INTRODUCTION

Beginning in the early 1960's through 1985 the USDA Agricultural Research Service collected detailed hydrologic data on the Little Washita River Watershed to determine the downstream hydrologic impact of USDA Soil Conservation Floodwater Retardation Structures (FRS). From 1969 through 1982 the SCS installed 45 detention reservoirs controlling about 45% of the drainage area. The effect of the FRS on water and sediment yield was investigated by the ARS (USDA-ARS, Staff, 1983) using a double mass plot (Searcy and Hardison, 1960) with an untreated neighboring watershed. The data from 1961 through 1978 showed no significant change due to the FRS (USDA-ARS, Staff, 1983), even though it is known that the reservoirs trap between 75% and 90% of the incoming sediment (Sharpley et al, 1989). The results were found inconclusive because: (1) the post-construction period was not included in the analysis; (2) the FRS affect peak runoff rates more than water yield; and, last but not least, (3) the relationship between cumulative yield and time (double-mass plot) is not sensitive to small and short duration changes, such as the impact of the FRS on sediment yield for so few years. A subsequent numerical model investigation (Nicks et al, 1985) showed that the FRS reduced water yield by 3% and sediment yield by 29%.

The present study addresses the existence and effect of two major climate fluctuations on the water and sediment yield of the Little Washita River Watershed. Because watershed runoff is considered, climate fluctuations are defined in terms of annual precipitation pattern. The annual precipitation pattern of the two climate fluctuations are compared to previous fluctuations, and their impact of watershed runoff is presented. The implications of the climate fluctuations on the previous sediment yield investigations are briefly addressed, and the effectivness of the FRS on reducing downstream sediment yield is estimated using sediment-discharge rating curves.

WATERSHED AND CLIMATE DATA

The Little Washita River Watershed is located in Southwest Oklahoma as shown in Fig. 1. The watershed drains 610 sq.km. of the Washita River Basin. Total drainage area upstream of gage 522 is 538 sq.km., and the drainage area affected by the FRS is 276 sq.km. or 45% of the watershed area (USDA-ARS, Staff, 1983). The land use is 20% cultivation, 65% range, and 15% miscellaneous uses such as timber, rock, urban areas, and reservoirs. No significant change in land use was reported after 1962 (USDA-ARS, Staff, 1983; personal communication by G. Coleman). Mean annual precipitation over the watershed is 750 mm. Most of the precipitation occurs in the spring and fall months.

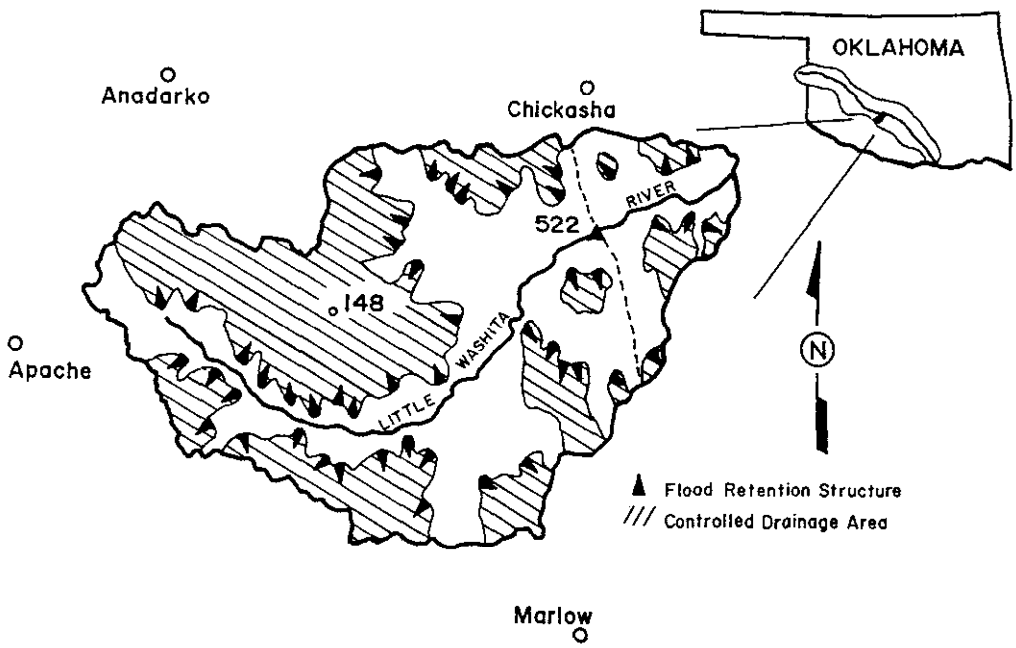

Fig. 1. Location map of the Little Wahita River Wastershed, Oklahoma.

Daily discharge and total sediment load at gage 522 were collected from 1963 through 1985. For the same period, daily rainfall data are available for 33 USDA-ARS raingages located in the watershed. Four additional U.S. Weather Bureau stations with long-term records are situated just outside the watershed boundaries at the four corners of the watershed as shown in Fig. 1. The stations are: Apache (34 0224 08; 1953-74), Chickasha Exp. St. (34 1750 05; 1953-89), Anadarko (34 0244 07; 1948-89), and Marlow (34 5581 08; 1901-89); numbers in parentheses are station identification and record length. The U.S. Weather Bureau stations and the raingages within the watershed are used to determine the annual and long-term average annual precipitation at the center of the Little Washita River Watershed. A single raingage, number 148, which is situated close to the center of the watershed, is used to analyze the statistics of point rainfall amounts and frequency of occurence.

RESULTS

Climate fluctuations

For this study, a climate fluctuation is defined as a period of predominantly above or below mean annual precipitation. While not presented here, a time series analysis of annual precipitation at the center of the Little Washita River Watershed, and corresponding cumulative precipitation for years 1901 through 1989 give no indication of any significant climate fluctuations. However, the cumulative deviations of the annual precipitation about the long-term mean show extended periods of above or below average annual precipitation as illustrated in Fig. 2. An upward or downward trend in data indicates consecutive years with above or below average annual precipitation, respectively. The steepness of the trend is a measure of the magnitude of the deviation about the long-term average annual precipitation.

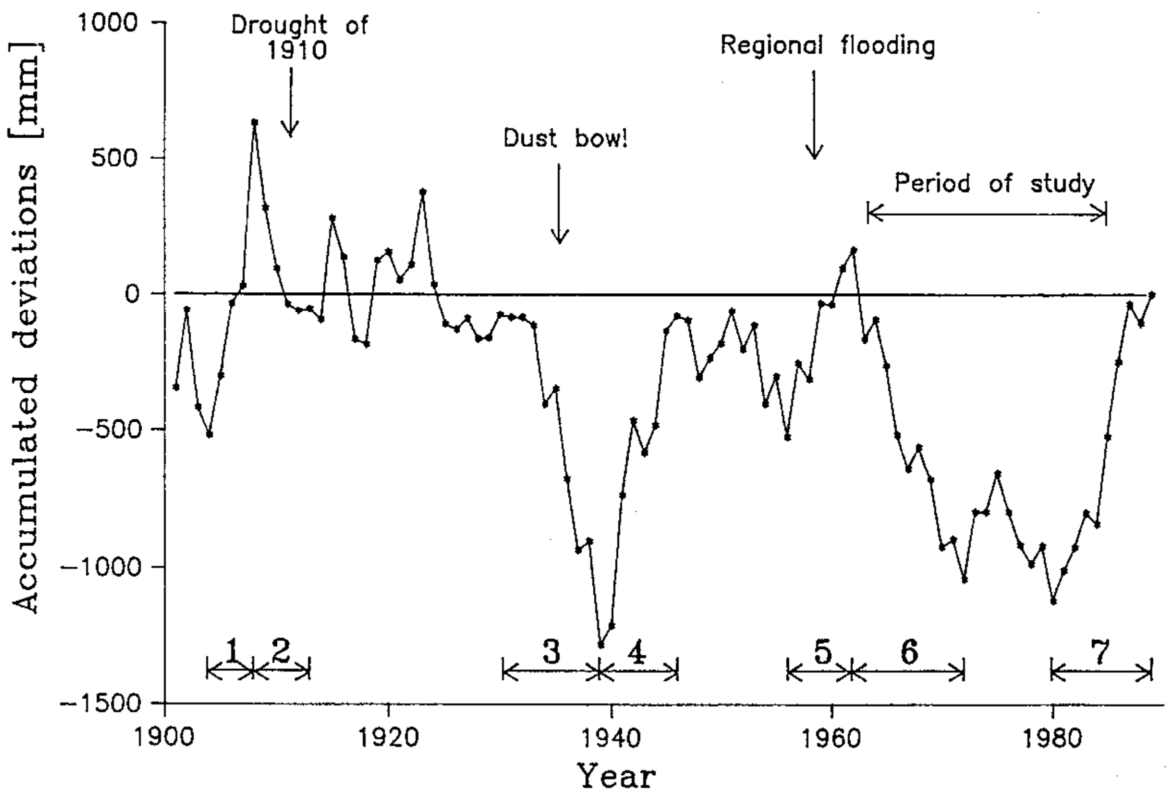

Fig. 2. Annual precipitation for 1901-1989 at the center of the Little Washita River Watershed, Oklahoma.

Seven major climate fluctuations are identified between 1901 and 1989: (1) the wet years of 1905-1908; (2) the drought of 1909-1912 (Griffits and Ainsworth, 1981; Ludlum, 1971); (3) the drought of 1933-1939, centered in the four corners of Oklahoma, Kansas, Texas and Colorado, referred to as The Dust Bowl (Rosenberg, 1978; Dale and Aldrich, 1969); (4) the wet years of 1940-1946; (5) the wet years of 1957-1962; (6) the dry years of 1963-1972 (Ludlum, 1971); and, (7) the wet years of 1981-1989. In addition, two smaller climate fluctuations occured between 1973-1980. Such climate fluctuations are a recurring phenomena (USDA, 1938; USDA, 1941). Climate fluctuations 6 and 7 affected the water and sediment data record of the Little Washita River Watershed. The two fluctuations are significant: the mean annual precipitation pattern of the dry years of 1963-1972 is similar to the one of The Dust Bowl in the 1930's, and the pattern of the wet years of 1981-1989 corresponds to the one of the early 1940's.

For the purpose of this study two periods are of particular interest. The first period represents the pre-reservoir (FRS) construction period. It begins in 1963, the year for which water and sediment data at gage 522 are available, and ends in 1969 when FRS construction was started. The second period corresponds to the post-reservoir construction period. It begins in 1981, the year in which 97% of the FRS were completed, and ends in 1985, the year in which the water and sediment data collection at gage 522 was discontinued. The remainder of this paper focuses on these two periods, which are referred to as the pre- and post-construction periods.

Annual variations

The annual averages of mean daily runoff and sediment yield follow a similar trend as the annual precipitation. This is shown in Fig. 3 for the 1963-1985 period. The pre-construction period lies entirely in the 1963-1972 drought, and the post-construction period entirely in the 1981-1989 wet years. During the pre-construction period, the mean annual precipitation was 670 mm, the average daily discharge was 0.53 cms and the average daily sediment yield was 425 mt/day. During the post-construction period the corresponding values are 910 mm, 1.65 cms, and 896 mt/day, corresponding to an increase of 35%, 210%, and 110%, respectively. The disproportionally larger increase in discharge and sediment yield are due to the nonlinear relation between precipitation, runoff and sediment. The implications of these changes in watershed runoff with respect to the evaluation of the downstream impact of the FRS is discussed later.

Frequencies

The mean number of rainy days in each month and the mean monthly rainfall are shown in Fig.4a and 4b. In general, both the mean number of rainy days and the mean monthly rainfall, are higher in the post- than in the pre-construction period. The mean number of rainy days per year increased from an average of 68 to 76, or by 12%. The largest increase seems to occur in spring and late fall. The average size of individual rainfall events increased from 9.6 to 11.4 mm, or by 18%. The increase in number and size of the rainfall events are the primary reason for the large increase in watershed runoff.

The frequency of daily mean discharge under 25 cms is shown in Fig. 5. Low flow events under 0.5 cms represented 77% of the runoff in the pre-construction period, whereas they only represented 33% in the post-construction period. On the other hand, flow events above 1.00 cms represented only 6% of the runoff in the pre-construction period, and 35% in the post-construction period. In the seven years of the pre-construction period nine events exceeded 500 cms, whereas in the five years of the post-construction period 29 such events were recorded. A similar trend is also observed for the daily sediment yield data.

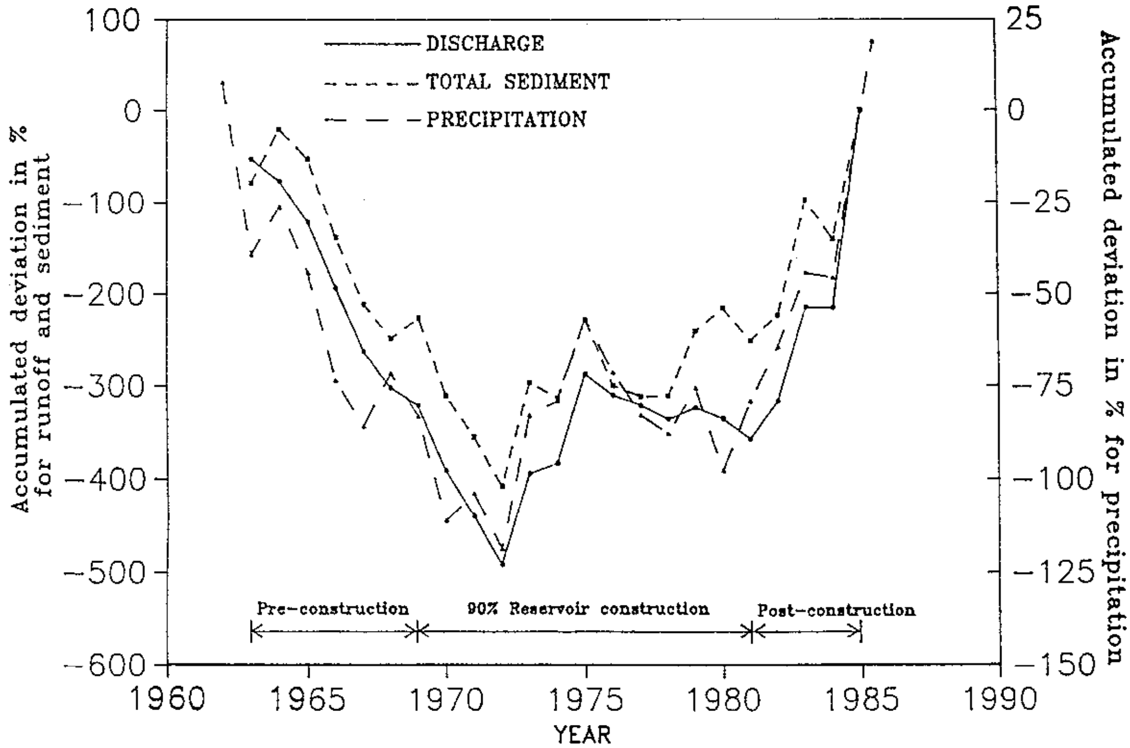

Fig. 3. Time series of annual precipitation at the center of the watershed, and annual averages of mean daily runoff and sediment yield for 1963-1985 at gage 522.

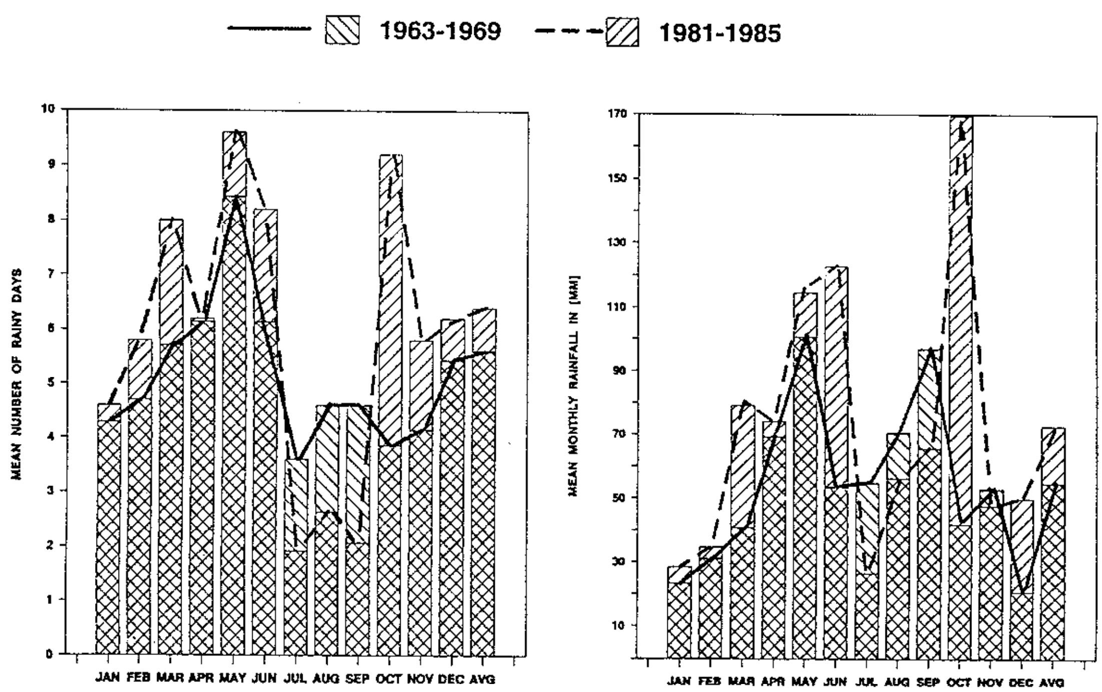

Fig. 4. Mean number of rainy days per month and mean monthly rainfall for the pre- and post-construction periods at raingage 148.

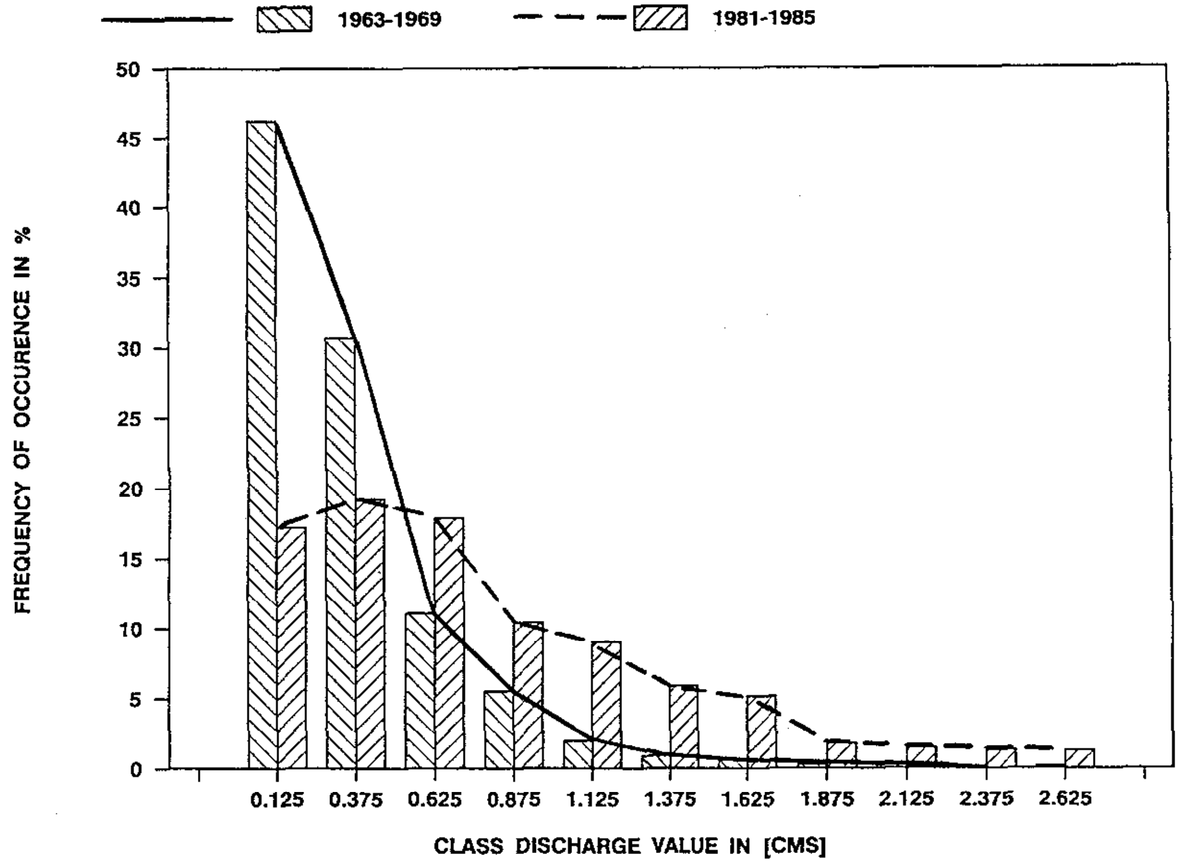

Fig. 5. Frequency of occurrence of mean daily discharge under 25 cms for the pre- and post-construction periods at gage 522.

DISCUSSION

The annual precipitation record shows that significant climate fluctuations affected the runoff of the Little Washita River Watershed between 1963 and 1985. Annual watershed runoff and sediment yield display a pattern similar to that of precipitation, an indication of the dominant influence of precipitation on watershed runoff. As a result of the transition from a dry (1963-1969) to a wet (1981-1985) period, water yield tripled, sediment yield doubled, and peak flows above 500 cms increased by a four fold in spite of the FRS. Frequency of occurence and amounts of precipitation, associated with corresponding changes in antecedent moisture conditions, are the main cause for the change in watershed runoff. The effects of the FRS, mainly a reduction in peak flows and in sediment yield, are overshadowed by the effect of the climate fluctuations. Therefore, the direct use of the observed data to evaluate FRS benefits is inappropriate and may lead to incorrect conclusions. The inconclusive findings of the USDA-ARS Staff investigation (USDA-ARS, Staff, 1983) with respect to the effect of the FRS on the water and sediment yield in the Little Washita River Watershed do not imply that downstream improvements did not occur, but merely that the length of the data record for the post-construction period was too short, and that climate fluctuations may have overshadowed the effects of the FRS.

Water and sediment yield are not easily normalized with respect to precipitation due to the strong dependence of runoff on antecedent moisture conditions and to the nonlinear relation between the rainfall-runoff variables. Relative comparisons between water and sediment yield may be more appropriate, because they are closely correlated and the effect of climate fluctuations is, to a large degree, eliminated. The impact of the FRS on sediment yield is estimated by comparing sediment yield at a given discharge for the pre- and post-construction periods. The rating curves of mean monthly discharge and sediment yield for the two periods are shown in Fig. 6. It appears that the FRS reduced the sediment yield by about 50%. This is consistent with a previous numerical model study by Nicks et al (1985) which shows a 29% reduction in sediment yield. The rather simple sediment routing component of this numerical model may account for some of the difference in sediment yield reduction. Not all the reduction can be attributed to the FRS alone because climate fluctuations impact soil cover, runoff, sediment erosion, and FRS operation characteristics. A disaggregation of the various cause-effect relations may be best achieved by a complementary use of physically based numerical models.

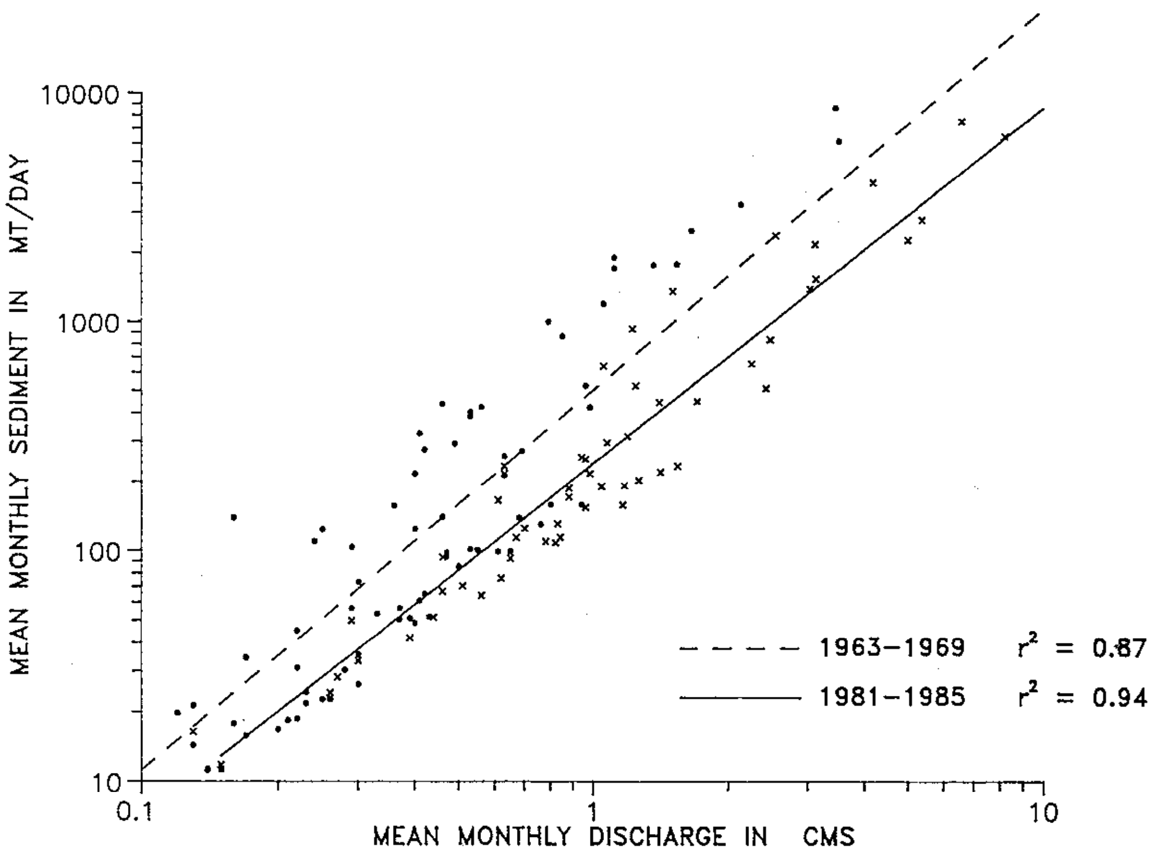

Fig. 6. Sediment-discharge rating curves for the pre- and post-construction periods.

CONCLUSIONS

Two climate fluctuations between 1963 and 1985 have a pronounced effect the on water and sediment yield of the Little Washita River Watershed. The magnitude of the change in watershed runoff can easily mask the beneficial effects of soil and water conservation measures. Therefore, measured changes in water quality may not necessarily be attributed to conservation measures. On the other hand, a lack of change does not imply that conservation measures are ineffective. Relevant hydrologic and meteorologic influences must be checked and, if necessary, adjustments must be made to avoid misinterpretations. The effect of climate fluctuations on sediment yield can be eliminated, to some degree, by analysing sediment yield as a function of runoff. A comparison of sediment yield at a given discharge showed that the FRS may have reduced sediment yield by 50%. A better disaggregation of the interdependent effects of the FRS and climate fluctuations on water and sediment yield may be provided by the complementary use of physically based numerical models.

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OCEAN DISPOSAL OF DREDGED MATERIAL: PLUME ANALYSIS

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ABSTRACT

The U.S. Army Engineer District, Jacksonville (SAJ), has proposed a new open-water disposal site for dredged material from Tampa Harbor, Florida. This site, located approximately 17 nautical miles from shore, would receive 3.6 million cu. yd. of dredged material from the Tampa Harbor channel deepening project plus an estimated 0.4 million cubic yards per year of maintenance dredging. SAJ was interested in determining the short and long-term fates of the sediment disposed at the site. This paper addresses the assumptions and methodologies that were used in determining only the short-term fate of the disposed dredged material. Short-term fate refers to the formation of the sediment plume (turbidity plume), the extent of its spread, and the expected depth of deposits before the sediment is resuspended by natural forces and leaves the general area. A key study component is the development of an exceedance frequency relation of the combined wave and current induced bed shear stresses. This relation is used to estimate the longest "quiescent" time period that the shear stress would not suspend the sediment, leading to the maximum expected accumulated sediment depth.

INTRODUCTION

The U. S. Army Engineer District, Jacksonville (SAJ), is proposing a new open-water disposal site for Tampa Harbor dredged material. This site, shown in Figure 1, would receive 3.6 million cu. yd. of dredged material from the Tampa Harbor channel deepening project plus an estimated 0.4 million cu. yd./yr of maintenance dredging. Approximately 0.7 million cu. yd./yr will be placed in diked areas with an expected life of 25 to 30 years. Presently, there are smaller existing diked and ocean disposal sites being used. The purpose of this study is to predict the short-term fate of the sediment material entrained during open ocean disposal operations. Short-term fate refers to the formation of the sediment plume (often referred as a turbidity plume), the extent of its spread and expected depth of deposits before the sediment is resuspended by natural forces and leaves the general area. Only a small percent of a dump is of sufficiently small particles to be entrained in a plume. The scope of this study includes all energy forces acting at the proposed site but is limited to the physical properties of the sediment material and not its chemical or water quality properties. The approach combines field observations at the proposed site, at the existing disposal site, and at other disposal sites. The scope also includes analytical techniques for calculating the size and movement of a sediment plume, the size of the dynamic collapse zone, and the resuspension of the deposited sediment.

Information in the following was obtained from EPA (1982). The proposed site, designated Alternative Site 4, is 17 nautical miles (n.m.) from shore and is square shaped with lengths of 2 n.m. on each side covering an area of 4 sq. n.m. The continental shelf gradient is generally 0.0005. Water depth at the site is 72 feet with very little variation. The sea bottom consists mostly of gently rolling sand with occasional shell fragments. Irregularly, the bottom has a thin veneer of unconsolidated fine sediments. No shipping lanes are near enough to affect operations. Currents in this area have two distinct seasons - summer and winter. Circulatory currents are generally southward in the winter and northward in the summer and tidal currents are generally in east-west directions. Monthly average current speeds are 10 cm/sec and are influenced mainly by detached cyclonic eddies (fluctuating from 10 to 30 cm/sec) from the Gulf loop current and by wind inducement. Although bottom currents are generally parallel to the surface currents, occasionally a vertical shear of 180 degrees can occur.

The dredging season is generally from February/March to August. For the channel deepening project the material is to be collected by clamshell and loaded into dump scows having a 3,100 cu. yd. capacity. The actual load is approximately 2,100 cu. yd. The maintenance material will be handled by clamshell, scow equipment or by hopper dredge. Within a 24 - hour period, 18 hours is usually the actual operation period with 6 hours for maintenance and idle time. Dredging is scheduled as needed and is sometimes discontinued in response to environmental considerations such as migration of marine animals through the disposal area. Disposal activities are halted during significant meteorological events such as frontal systems, tropical storms, and hurricanes and are resumed when conditions are favorable for disposal operation.

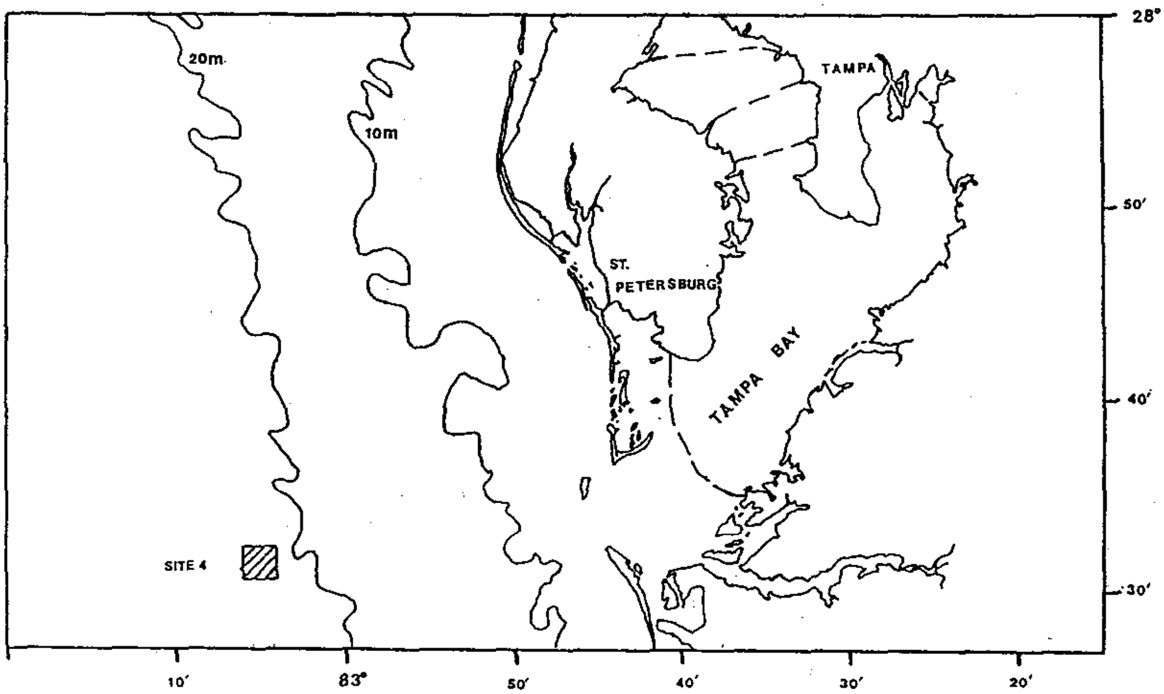

Figure 1. Location of proposed Alternative Site 4.

The material to be disposed has a bulking factor of 1.0 - 1.2 (ratio of volume of water-sediment mixture to volume of sediment). A bulking factor of 1.2 corresponds to a percent moisture content (PMC) of 50% (weight ratio of water to sediment X 100 percent) assuming a specific gravity of sediment of 2.5 relative to seawater. Although sediment gradation can vary, depending on dredging location, the dredged material can generally be expected to consist of 65% coarser than 0.062 mm, 5% between 0.062 mm and 0.004 mm, and 30% finer than 0.004 mm. The writer visited Site 4 and inspected samples of the bottom sediments obtained by divers. The samples generally consisted of medium to coarse sand with a small portion of shell fragments. EPA (1982) has also collected bottom samples at Site 4 and gradations have been published. The writer went to EPA in Washington, D. C. and viewed their video tapes of the ocean floor at the existing sites and Site 4. These tapes were made by dragging a camera-mounted sled in a uniform crisscrossing pattern across the sites. The basic bed characteristics are large areas of sand ripples and small dunes, occasional rock outcrops, and the unconsolidated fines in the dune troughs. This agreed with the descriptions given by EPA (1982).

OCEAN BED CONDITIONS

Wave data summarized in the Navy Oceanographic Atlas (U.S. Naval Oceanographic Office, 1963) for the Gulf north of 25 N and east of 85 W show that moderate conditions exist over this area most of the time. Ocean currents at Site 4 are produced by four energy mechanisms: wind stress, the Gulf loop current, tides, and tropical storms. The tidal current are generally less than 1 knot. Tropical storms and hurricanes produce strong bottom currents (3 to 4 knots), which disturb the natural as well as disposal material at Site 4 (EPA 1982).

The suspension of bottom sediments is related to the magnitude of the bed shear stress. Both currents and waves are significant, i.e., oscillatory fluid motion resulting from surface waves exerts shear stresses on the bottom that are often several times larger than shear stresses produced by unidirectional currents of the same magnitudes. Also, the bed shear stress produced by wave motion puts sediment particles into suspension where they can be transported by currents which normally would not be of sufficient magnitude to initiate sediment motion. Bed shear stress is related to bed shear velocity as follows:

$$\tau_b = \rho u_*^2 \tag{1}$$

where: τ_b = bed shear stress
ρ = water density
u_* = shear velocity

For this study a Bijker-type equation (McAnally and Thomas, in preparation) for total shear velocity caused by both waves and currents was used:

$$u_* = \tfrac{1}{2}(f_c u^2) + \tfrac{1}{4}(f_w U_{om}^2) \tag{2}$$

where: f_c = shear stress coefficient for currents, 0.003 (Sternberg 1972)
u = current velocity
f_w = shear stress coefficient for waves
U_{om} = maximum orbital velocity of waves

To evaluate this equation, it is necessary to know both shear stress coefficients and both velocities. Observations of wave heights and periods from 29 years of wave data (U.S. Naval Weather Service Command 1970) at Fort Myers (25° to 27° N and 81° to 83° W) were used to help develop a percent exceedance curve for wave-induced shear stresses. Using the method in Beauchamp (1974), the maximum orbital velocities, U_{om}, were calculated as follows:

$$U_{om} = (\pi H/T)/\sinh(2\pi d/L) \tag{3}$$

where: T = wave period
H = wave height
d = water depth

From March 9 to May 12, 1983, the U.S. Army Engineer Waterways Experiment Station (WES) and SAJ deployed recording current meters at the disposal site. ENDECO 105 current meters were placed at two locations, approximately 1 mile part, oriented north-south in the center of Site 4. Meters located at the center of the site were placed at 3 and 9 feet from the bottom. Another pair of meters, located 1 mile south of the center, was placed at 3 feet and mid-depth. From these data sources, the current velocities were plotted on probability paper forming a percent exceedance curve for current velocity. Assuming the same exceedance frequency for current and orbital velocities, then for a given exceedance frequency, the current velocity and orbital velocity can be obtained. Using Equation 2, the total bed shear stress can be calculated for that frequency. The orbital and current velocities, bed shear stress, and associated frequencies are shown in Table 1. The frequency relation assumption between current velocities and wave energy is more conservative (i.e., a greater scour potential) than the assumption of an independent relationship. The highest currents (4 knots) are associated with hurricane conditions which have occurred about 1 percent of the time (EPA 1982).

TABLE 1 Frequency of Shear Stress and Current Velocity

Exceedance Frequency, Percent	Current Velocity, ft/sec	Orbital Velocity, ft/sec	Bed Shear Stress, $lb/ft^2 x10^{-3}$
99.0	--	--	0.000
87.1	0.17	0.09	0.216
59.2	0.20	0.13	0.313
28.5	0.33	0.40	1.000
11.0	0.55	0.68	2.334
8.3	0.75	0.80	2.988
6.6	0.90	0.97	4.033
4.0	1.30	1.37	7.826
2.9	1.60	1.70	11.245
2.2	2.00	2.55	17.333
2.0	2.20	2.89	20.548
1.8	2.30	3.24	22.520
1.6	2.50	4.11	27.185
1.4	2.75	5.11	33.394
<0.6	6.79	5.77	143.537

DISPOSAL OPERATION PHENOMENA

When the hopper doors are opened, the dredged material behavior is described in three phases: convective descent, dynamic collapse, and passive diffusion. The convective descent is characterized by the dredged material acting as a distinct sediment fluid mixture falling under the influence of gravity (negative buoyancy). As this cloud descends, a vortex ring occurs in which the central region of the cloud moves downward and the outer region moves upward with a net movement downward. The downward motion of the central region tends to maintain the cloud as a unit; however, the upward motion of the outer region enhances the entrainment of ambient water into the cloud which aids in the dispersal of the material before it reaches the bottom. The short-term fate of the material entrained by this phenomenon is analyzed in a later section. Dynamic collapse occurs when the cloud, as it spreads during descent, either reaches the bottom or the density difference of the cloud and ambient water is small enough to achieve a buoyant state. The vertical descent of the cloud is stopped and horizontal spreading occurs. This sets the stage for the settlement of particles if dynamic collapse occurs above the bottom. When the horizontal spreading during dynamic collapse reduces to an order of magnitude of spreading due to turbulent diffusion, passive dispersion becomes the predominant mechanism of sediment movement. Passive dispersion is important only when the dynamic collapse occurs above the bottom. When this occurs, sediment particles settle according to their fall velocities resulting in deposition of large particles near the disposal position and smaller particles farther out.

FATE OF SEDIMENT PLUME

Due to the nature of the clamshell and hopper dredge operation, the dredged material is very dense with a PMC of 50 indicating characteristics similar to a bulk solid. Tests performed by the JBF Corporation (JBF Scientific Corporation 1975) have shown that during disposal, very little entrainment of material occurs in either silt or clay for a PMC up to 100. Table 2 describes the behavior of sediments in the solid and liquid modes. This indicates that disposal operation will impart very little material to the water column, i.e., a small sediment plume, and dynamic collapse will occur at the bottom. To determine the sediment plume's fate, the amount of material entrained by the water must be determined. Gordon (1974) concluded that less than 1 percent of the dredged silt in the dump was entrained. Bokuniewicz et al. (1978) found that less than 5 percent is generally lost during disposal operation and Tavolaro (1982) found that an average of 4 percent was lost during operatic outside New York Harbor. Based upon the above and a PMC of 50, 2.5 percent of the dump estimated to be entrained as a sediment plume during disposal operations at Site 4.

TABLE 2 Comparison of Properties of Disposal Material in Solid and Liquid Modes (JBF Corp. 1975)

Solid Mode	Liquid Mode
1. Low PMC.	1. High PMC
2. Observed in barge filled by clamshell dredge.	2. Observed in material of the upper few feet of hopper dredge (much affected by transit time).
3. Usually found in the bottom half of hopper dredge	3. Characteristic of pipeline dredge material.
4. Falls as solid blocks.	4. Falls as liquid cloud.
5. Rapid descent, no deceleration.	5. Slower descent phase.
6. Little cloud growth before impact.	6. Cloud expands due to entrainment.
7. Trails a small turbidity plume.	7. Deceleration of descent is significant.
8. Little spread of material on bottom after impact, depends upon cohesiveness of material disposed.	8. Horizontal momentum on bottom considerable, producing laterally spreading cloud.
9. Pycnocline has effect only on small trailing turbidity plume.	9. Pycnocline has effect on falling cloud, possibly producing first collapse.
10. Generally some mounding on bottom, even in deep water.	10. Little or no mounding on bottom.

A computer code developed under contract for WES to study sediment (turbidity) plumes was used to trace the short-term fate of the dumped material. This code, described in detail by Wechsler and Cogley (1977), predicts the downstream concentration gradient of silt and colloidal-size fractions of dredged sediments discharged in waters characterized by unidirectional constant flow, essentially infinite width, constant depth and infinite length. Density gradient settling, salt wedges, narrow channels, tidal flows and complex circulation patterns are beyond the model's scope. For an average load of 2,100 cu yd, a bulking factor of 1.2, and 2.5 percent entrainment, the plume sediment volume can be calculated by (2,100 yd^3/1.2) x 0.025 x 27 ft^3/yd^3 = 1,180 ft^3/load. Since the computer code requires a constant sediment load rate, the load from one scow was assumed to be spread over a time equal to the round trip time for one load (6 hours). The equivalent load rate then becomes 0.055 ft^3/sec. Vanoni (1977) showed that the initial specific gravity of deposited sediment found by various researchers ranged from 86 lb/ft^3 for 0.062 mm size particles to 55 lb/ft^3 for 0.004 mm size particles. Because 85.6 percent of the plume is expected to be clays, an average density of 60 lb ft^3 was adopted for the specific weight of the sediment.

The grain sizes that would be in the sediment plume consist primarily of clays and silts (less than 0.062 mm) and is 35% of the total load. Larger particles have higher fall velocities such that they would readily deposit in less than 3 hours at a 72 foot depth and would travel less than 900 feet at an average current velocity of 0.25 ft/sec. If 2.5% of the material is assumed to be entrained, and that the silts and the clays are the only size fractions to be entrained, 7.1 percent of the silts and clays will be in the sediment plume and the rest will reach the bottom as colloidal clumps or aggregates. This phenomenon is typical of material obtained from clamshell and hopper dredge operation (Pequegnat et al. 1978). Utilizing five size classes, the fall velocity of each class was obtained from column settling tests of Hillsborough Bay borings (USAED, Jacksonville 1975). The tests were performed using 20 parts per thousand (ppt) salinity solutions at 23°C. The disposal site salinities ranged from 31.1 to

37.5 ppt (EPA 1982). Actual clay and silt fall velocities would be higher than the laboratory results because the higher salinity and vertical velocity gradient would enhance aggregation which in turn creates larger effective particles with higher fall velocities.

Water movement measurements obtained from the center of Site 4 at the 3 foot depth were plotted on a rose chart and were generally omni-directional. The time-weighted current velocity at the center of Site A was 0.226 ft/sec (6.9 cm/sec) and is comparable to the monthly average of 10 cm/sec (0.328 ft/sec) mentioned in EPA (1982). Since the monthly average included higher currents from winter storms (no disposal operations), 0.25 ft/sec (7/.6 cm/sec) current velocity was considered most representative of disposal operations conditions. The water depth was assumed to be constant at 72 feet. The low PMC material would stay relatively intact as it is dumped until it reaches the lower one-third depth. In the model, the material was released at a vertical water zone of 8 to 24 feet from the bottom. The initial plume diameter in this zone was assumed to be 120 feet, which is comparable to plumes calculated by Johnson and Holliday (1978) for the New York Bight disposal operation in 83 feet water depths. The computational grid extended 15,000 feet in 300 foot increments from the disposal site and was 1,200 feet wide in 40 foot increments.

Since no prototype lateral dispersion rate information was available for model calibration, the results were compared with values found in the literature. Brooks (1960) defined turbulent dissipation parameters from 0.00015 for open seas to 0.0005 for a dynamic estuary. A turbulent dissipation parameter value of 0.00022 was used in the model which is expected for open seas of moderate depths. Wilson (1979) computed diffusion velocities for plumes in seawater up to 10 feet deep and found them to be about 1 cm/sec. The computed diffusion velocity for the model is 0.38 cm/sec. Because the water depth for the model is 72 feet, the lower diffusion value is in the range expected.

If the disposal occurs at the site's center, the shortest distance to the site limits is 1 n.m. The calculated average plume concentration at 1 n.m. from the center is 3.6 mg/1 with a maximum concentration of 20.5 mg/1. The plume is approximately 720 feet wide at this location. When the concentration in the sediment plume drops to the same order of magnitude as the ambient concentration, the water can then be considered "unaffected" by the disposal (Christodoulou, Leimkuhler, and Ippen 1974). EPA (1982) found the natural suspended particle matter at Site 4 to be 0.55 to 2.97 mg/l, thus when the plume concentration reached 2 mg/l during disposal, it was considered to be background. The disposal simulation showed that the average concentration reached 2 mg/l at a 15,000 foot distance from the site center with a peak of 12.8 mg/l at the plume's center.

Calculations were made to determine the maximum deposit depth from the sediment plume before the deposits are suspended. Mantz (1977) found that a critical bed shear stress of 0.0016 lb/ft^2 would initiate the movement of a deposited material in the silt range (0.03 mm) and Krone (1962) found the critical bed shear stress to range from 0.0008 to 0.0014 lb/ft^2 for San Francisco Bay mud (silt and clay) deposited at concentrations of less than 300 mg/1. Using a relatively strong value of 0.002 lb/ft^2 for the critical stress of the sediment deposited from the sediment plume, the shear stress can be expected to be exceeded 12 percent of the time (see Table 1) or about 44 days out of a year. Consequently for about 320 days of the year, but not necessarily consecutively, the ocean conditions are sufficiently mild such that deposits from the plume will not be resuspended and will accumulate. The probable maximum consecutive number of days for such conditions is 60 days and was determined by analyzing monthly wave observations and determining the maximum length of time the combined wave/current bed shear is less than the critical value for resuspension. When conditions are sufficiently severe to resuspend, it is assumed that the total depth of the plume deposit is resuspended and that the currents will completely mix the sediments with the natural background sediment concentration turbidity in the water column. The currents are generally omni-directional with the longest duration in one direction quadrant being about 3 days. Based upon this observation, the 60 day plume deposits are distributed radially from the center of the disposal site to determine the maximum depth of deposits due to the sediment plume. Table 3 shows the results of the analysis for a single disposal and for continuous disposal operation (i.e., 1 dump every 6 hours) for 60 days.

TABLE 3 Sediment Distribution of Deposits from a Sediment Plume

Distance from Disposal Point, ft	Width of Plume for One Disposal	Avg. Depth of Deposits, mm	Maximum Depth of Deposit, mm	Avg. Deposit Depth after 60 days, mm
1,500	480	0.0472	0.260	1.15
3,000	560	0.0117	0.064	0.125
4,500	640	0.0070	0.039	0.0466
6,000	720	0.0047	0.026	0.0244
7,500	800	0.0034	0.018	0.0152
9,000	920	0.0026	0.014	0.0105
10,500	920	0.0021	0.012	0.00771
12,000	960	0.0018	0.010	0.00601
13,500	1,000	0.0016	0.0089	0.00485
15,000	1,040	0.0015	0.0080	0.00405

Approximately 35 cu. yd./dump is expected to leave Site 4 in the sediment plume. This translates to 2 percent of each dump. Krone (1962) has shown that "effective" fall velocities can increase an order of magnitude due to flocculation of clay and silt particles under chemical and physical action; therefore these results can be considered to be under maximum probable conditions with the actual quantity of sediment leaving the site somewhat lower than shown. Although the relationships between loading rates, concentration, and depth of deposits are not exactly linear, a good approximation of concentrations and depths of deposit for different loading rates can be made by taking the ratio of the load rates and applying it to the concentrations of depth of deposits. For example, if there were two scows being used and disposal occurred every 3 hours. (double the load rate), the average concentration at the disposal site limits would be 7.2 mg/1 (2 x 3.6 mg/1) with a 60-day depth of 0.0488 mm (2 X 0.0244 mg/1 from Table 3) at 6000 feet from the center. The width of the plume stays essentially the same, regardless of the concentration.

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