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14. ABSTRACT
A seminar on Sediment Transport in Rivers and Reservoirs was held on 7 - 9 April 1970 at the Hydrologic Engineering Center (HEC) in Davis, California. The seminar was held primarily to exchange information with the Corps of Engineers on techniques used in, and problems connected with, the prediction of sediment transport in rivers and reservoirs.

The purpose of the seminar is to describe problems of sediment transport, techniques that are currently in use for evaluating sediment transport, and the potential for applying computer technology in devising better solutions to these problems. In keeping these three points in mind the discussions on each paper will particularly concentrate on the potential for improving techniques and for obtaining better solutions to the major sediment transport problems.

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FOREWORD

This seminar was held primarily to exchange information within the Corps of Engineers on techniques used in, and problems connected with, the prediction of sediment transport in rivers and reservoirs.

Presentations are, in general, frank evaluations by the authors and are not official Corps documents. The views and conclusions expressed are those of the individual authors and are not intended to modify or replace official OCE Engineer Regulations, Engineering Manuals or Engineer Technical Letters.

It is hoped that the presentations included herein will help to define and illustrate the "state of the art" for the benefit of those endeavoring to advance technology in the field of sediment engineering.

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PAPER NUMBER

SEMINAR ON SEDIMENT TRANSPORT IN
RIVERS AND RESERVOIRS

INTRODUCTORY REMARKS

by

LEO R. BEARD, Director
The Hydrologic Engineering Center

I want to welcome all of you to The Hydrologic Engineering Center and to this seminar on Sediment Transport in Rivers and Reservoirs. As you all well know, the many factors affecting sediment transport are complex, and the techniques of measuring and evaluating sediment transport leave much to be desired. Among the many factors affecting sediment transport, the rate of streamflow and the rate of change of streamflow are of major significance. Since these vary from day to day, and since their effects are non-linear, correlations are obscured, and it is difficult to devise simple parameters of streamflow that can be used effectively for evaluating sediment transport in the field. As a consequence, derivations of empirical relationships have been greatly handicapped because of the great variation of flow that occurs in most streams.

The ability to analyze large amounts of data by use of electronic computers may provide a means of deriving more definitive relationships between sediment transport and all of the physical factors involved and of calculating sediment transport quantities. Many of us in The Hydrologic Engineering Center feel that it should be possible to construct rational mathematical models, based on existing knowledge of the many factors involved. These factors include characteristics of the sediment load, slopes and cross sections of the streams, sources of sediment, and many other factors. Once a reasonable computer model is devised, it can be calibrated to reproduce observed sediment transport quantities at various locations. Such a model must be capable of assessing aggradation and degradation at any time and location, depending on streamflow characteristics and character of the sediment load. This would include deposition of sediment in lakes and reservoirs. Eventually it would be desirable for such a model to evaluate scour and deposition at a large number of points under specified sequences of flows for long periods and under specified conditions of channel modification.

This, in general, would be an ideal hydrologic tool for the solution of those sediment engineering problems that are heavily dependent on hydrologic factors. As will be clear from the papers presented here, such a solution is in the distant future, if indeed it is feasible at all. At this stage, we feel that it is well worth pursuing.

In our original announcement of this seminar, we stated that the purposes are to describe problems of sediment transport, techniques that are currently in use for evaluating sediment transport, and the potential for applying computer technology in devising better solutions to these problems. I hope that you will all keep these three points in mind and that our discussions on each paper will particularly concentrate on the potential for improving techniques and for obtaining better solutions to the major sediment transport problems.

We hope that, while you are here, you will become acquainted with the staff at the Center and with the work that we are doing. If there is any way that we can help in regard to your accommodations or travel or other matters while you are here, please let us know.

BASIC SEDIMENTATION PROBLEMS

by

D.C. Bondurant[1]

Investigations of sediments and their effects on stream channel and reservoir projects have been conducted for more than 40 years. Much has been learned, but there is yet much to be learned. In fact, a list of problems on sediment transportation requiring much further study, compiled by Dr. V.A. Vanoni at the Inter-Agency Sedimentation Conference at Denver, Colorado, in 1947, is as valid today as it was then. Surprisingly, many of those associated either directly of indirectly with sediment problems, lack a real understanding of some of the basic sediment relationships. It is the intent of this paper to discuss briefly the basic relationships, to note the primary problems and the progress in the solution of these problems, and to suggest requirements for continued study.

The term "sediment" generically refers to any solid matter that settles to the bottom of a liquid. This discussion, however, will be restricted to "fluvial sediments" which are generally those associated with streams or bodies of water. They are normally earth materials eroded or transported by water or deposited from it. Except for some heavy minerals, sediments have a specific gravity averaging about 2.65, and range in size from sub-microscopic clay particles to large boulders. They are most often classified by physical size limits as clay, silt, sand, gravel, cobbles, and boulders; however, the characteristic of primary importance is the terminal velocity in still water at 24 degrees Centigrade temperature.

The size limits between the sand and larger sediments are purely arbitrary. The distinction between sand and silt sizes (0.0625 mm) coincides with a distinction in the mechanics of the fall in still water, and the division between silt and clay (about 0.004 mm or 4 microns (4μ)) represents the beginning of effective electro-chemical activity.

The terminal velocity at which a solid particle will fall through a fluid results from gravitational forces acting on the particle and the sum of the inertial and viscous forces resisting the movement within the fluid. These two latter forces are approximately equal for a quartz sphere of 0.0625 mm falling at terminal velocity in still water with a temperature of 24 degrees C. As the particle size decreases, the inertial effects become negligible, and as the particle size increases, the viscous effects become negligible. The result is that sediments finer than about 0.0625 mm will be evenly distributed throughout the flow of a stream, while the coarser material will be distributed in accordance with

[1] Chief, Sediment Investigation Section, Missouri River Division

the strength of the turbulence. The size of 0.0625 mm for the classification limit is, of course, an average, for, as the viscosity of the fluid increases or decreases from that of water at 24 degrees Centigrade, the proportional effect of the viscous forces increases or decreases accordingly.

Most sediment particles, due to their molecular constitution, will have a net ionic charge at their surfaces. If the particle is larger than about 0.004 mm (4μ) in diameter, the surface to mass ratio is too small for this charge to have any appreciable effect on the action of the particle; however, as the size decreases, the ratio of surface area to mass increases rapidly, and the effect of the surface charge becomes very important. In addition, the effect of mass attraction between the particles becomes more important. No attempt will be made to go into detail here other than to state that the ionic charge differs with varying minerals, and that there is an interaction between these ions and those which may be present in the fluid. The result is that, depending on the character of the mineral and the chemistry of the fluid, the clay particles may be grouped into clumps, or floccules, which have a fall velocity equivalent to that of a particle equal in size to the foccule, or they may be individually dispersed, or they may form a soft thixotropic mass. The latter is a condition wherein the mass acts as a solid until an extermal force is applied, whereupon it can flow as a fluid.

The flocculated clay settles readily; thus, it usually deposits near the head of a reservoir. It tends to fill the deeper portions of the cross section first, and the lateral surface of the deposit tends to be level. In the disperse phase, the clay may remain in suspension until it moves much further into the reservoir, and it is likely to deposit to a relatively even thichness across the entire section. Very fine disperse clays, in the colloidal size range, may remain in suspension for several days until the entire reservoir becomes turbid or cloudy. With the proper chemistry of the fluid and type of clay mineral, the clay suspension may form a density flow, or may deposit in a low density, thixotropic mass at the immediate head of the reservoir.

Sediment in a stream channel is classified either in terms of mode of transport or in terms of its presence in the stream bed. In the latter case, it is generally divided into the two classifications of bed material load and wash load. Bed material is that material found in appreciable quantities in the bed of the stream. It is also the material which has been or is being furnished by the watershed in quantities equal to or greater than the capacity of the stream to transport it. Only for this material can a balance be established between transport and transport capacity, and this balance is the basis for all transport formulae. I have been amazed at the number of workers, who should know better, who arbitrarily classify all sand size sediments and larger as bed materials, and how often transport formulae are utilized to estimate aggradation or degradtion based on sand size sediments found in samples of suspended sediments. In streams such as the Missouri River or the Mississippi River, less than 10 percent of the suspended sands may be of the sizes found in the bed. This will be discussed further later.

In 1947, Dr. Vito A. Vanoni, in a paper presented at the first Federal Inter-Agency Sedimentation Conference at Denver, Colorado, listed six problems of sediment transportation which were of importance and needed to be studied further. It will be well to state these problems and review the work that has been done on them to date. As presented, they were:

1. The laws governing the transportation of natural sand mixtures such as found in streams.

2. The factors governing the production of sand waves and their effect on the roughness of the channel and the flow characteristics.

3. Detailed studies of the turbulence distribution and the effect of the sediment on the turbulence.

4. Studies of the exchange coefficient for suspended sediment and the factors governing its variation.

5. Studies of secondary circulation and its influence on sediment transportation and stream behavior.

6. Measurements in natural streams to assist in extending the reliable range of equations for sediment transportation.

A cursory review of the matter seems to indicate that no progress in the development of a complete theory of transportation has been made since 1947. There has been progress, however, primarily in varied segments which cannot now be fitted into the whole.

In regard to the knowledge of movement of sediment as bed load, there has been no advance at all. Bed load cannot even be measured except by very elaborate installations suited only to a limited number of streams. In the laboratory, bed load is distinguished as the difference between total load and suspended load. In the field, it must be estimated. The Einstein bed load function remains one of the very few analytical tools available, other than very approximate empirical formulae, but it cannot be checked in most streams.

The laws of transportation of sediment mixtures, as a special function of the laws of transportation, have made no progress. It is doubtful that any investigators are working on the problem, except as associated with general transportation studies. In any event, there remains a size restriction regarding the size value that is supposed to be representative of the mixture. There are formulae in which the transport of each size fraction is computed, but these were generally available prior to 1947.

In general transport studies, a major advancement was made with the realization that it was not possible to relate the sediment transport of a stream to the cross-section and slope of the channel and the size of bed material, but that, when either the mean velocity and depth or water discharge and total sediment discharge were known, the remaining variables could be determined. This is apparently associated with the changing bed forms which cause large variations in the channel roughness. This knowledge created new problems that require further study. It is known that a high sediment transport engenders a smooth bed and that a low transport engenders a rough bed, but the knowledge available for practical use is more or less restricted to empirical relationships that involve grouping the various degrees of bed forms with various ranges of stream power. No one has yet been able to derive an analysis of the mechanics involved, although several approximations have been made.

The general effect of the fluid temperature has been reasonably well established, but finite relationships are not yet available. It is reasonably well established that, as the fluid temperature decreases, the suspended sediment transport increases and the bed roughness diminishes. Laboratory tests involving bed load movement only indicate that, conversely, the bed load movement decreases with decreasing temperature, but, again, the bed roughness continues to be greater with the lower sediment load. The problems of bed roughness, stream stage, temperature, and sediment load are being studied in the Missouri River, and some of these studies will be discussed by Mr. Warren Mellema.

In the problem of the exchange coefficient for suspended sediment, very little progress has been made. It has been learned that the Von Karman constant 'k', which is essentially a turbulence function derived from exchange theory and which is utilized in formulae for velocity distribution, sediment distribution in the stream vertical, and in transport formulae, is not constant except for the flow of clear water or other fluid. In a sediment laden flow the value of this function is not the same as for clear water flow, nor is it a constant. The value for velocity distribution differs from that for sediment distribution, and some field tests indicate that it differs in respect to different sediment sizes in a given flow.

It is known that in a sediment laden flow the logarithmic distribution of velocity and sediment is not constant from the surface to the bed; rather, there are two distinct zones. The lower zone, which may be as much as 20 percent of the total depth, transports the heavier load. There are only a few references available, however, describing this phenomenon, and they are not altogether definitive. There is a serious need for correlation of the available information and for further research.

It was demonstrated prior to 1947 that sediment suspended in the flow restricted the turbulent exchange; indeed, the velocity of a stream transporting a suspended sediment load may be as much as 25 percent higher than an equivalent clear water flow.

Most of the suspended sediment in the average stream, though, must be classified as wash load; thus, the question arises as to just how the wash load must be considered in transport computations. Most transport investigations in natural streams seem to have analyzed all suspended material of sand size or larger as bed material, and most seem to develop an average relationship which can be utilized with liberal judgement. It is difficult to accept such analyses when it is obvious that 90 percent to 95 percent of the material considered could not have been transported at capacity rate, although they might be adequate to compute the summation of the load in a vertical on the basis of one or more point samples.

In respect to rectification works on natural meandering channels, our criteria in the past were based on a few empirical observations. In recent years, however, there have been some very interesting investigations of the relationships between secondary currents and the formation of meanders. To date, these studies are mostly of a preliminary nature, but they appear to offer a very important path toward a more complete understanding of stream channel characteristics.

In summary, it appears that just enough has been learned since Dr. Vanoni's summation to create even more confusion. This is typical of research in any emerging discipline, and should not be regraded as failure; rather it should encourage researchers to review the situation and plan a coordinated attack. Substantial progress has been made in:

a. Knowledge of bed forms and their general relationships with the flow. It is known, for example, that a channel can adjust internally for almost a ten-fold variation in sediment load; the effect of temperature variations on sediment transport and on discharge characteristics can be appreciated, and a much better base from which to study transport now exists.

b. Appreciation of the effect of a heavy sediment load near the bed upon the distribution of sediment and velocity in stream flow.

c. Appreciation of the fact that there are really two zones of turbulent transport in a stream vertical rather than a single zone.

d. Appreciation of the role of secondary currents in the formation of a channel.

The major problem is that there does not seem to be any coordinated interest in continuing research on the subject. In 1947, Dr. Vanoni stated that there were probably less than 10 professional men in this country devoting a major part of their time to the study of the mechanics of sediment transportation. It is doubtful that he could count that many today, and most of the work that is being accomplished is being done with inadequate support by men who are engaged in phases of particular interest to them or for which they have developed ideas worthy of study. Anyone who doubts the current lack of interest can count the 15 papers listed to be presented here when a meeting of this type should logically have a generous Corps wide representation. If any progress is to be made toward a real scientific appraisal of the sediment problems involved in our channel and reservoir design, rather than continue with our more or less informed judgements, it will be necessary to make every effort to encourage a policy of a review of work being done, planning of investigations so that they will be well coordinated, and encouraging active support of the work.

SEDIMENTATION ACTIVITIES IN THE NORTH PACIFIC DIVISION

by

Billy J. Thomas[1]

The North Pacific Division is comprised of the Columbia River Drainage Basin, the coastal streams in Oregon and Washington, the Closed Basin in south central Oregon, and the State of Alaska. The drainage area in the contiguous U.S. (Chart 2) is 290,000 square miles and encompases all of the State of Washington, most of Idaho and Oregon, and parts of Montana, Nevada, Utah, and Wyoming.

The Cascade Range, running north and south through the center of Oregon and Washington, separates the basin into two distinct climatic regions. The generally wet west side has moderate temperatures and the more arid east side has it extremes of weather. The streams have fairly steep gradients and drainage is chiefly to the Willamette River and Puget Sound on the west side of the Cascades and to the Columbia River on the east side. These two climatic regions seem to be the main factors in the sediment loads carried by the streams in the Northwest. The program in each region will be discussed in this paper.

Extensive investigations of suspended sediment in streams have been made in the area west of the Cascade Range in Oregon. The first major investigation was made during the period between December 1948 and July 1951. At this time the Portland District initiated a suspended sediment sampling program at some twenty-one locations in the Willamette River Basin. The sampling stations were generally located near the mouths of streams tributary to the Willamette or just below the reservoirs, with two stations being located on the main Willamette. Whenever possible, the sampling points were located at or near gaged streamflow stations. The procedure followed in this investigation involved taking samples at least once a week from each of the stations and more frequently during the periods of high flows. Figure 1 shows typical sampling cycles for the program, with discharge hydrographs and graphs of sedimentation concentration and load. Table 1 lists the sampling stations together with their respective drainage areas, the number of samples, and the concentration of sediments for the entire period. Conclusions based on these data were that the sedimentation concentrations in streams west of the Cascades are low, ranging from 20 to 30 ppm in low flow periods and to 200 to 300 ppm in periods of high flow. The maximum concentration recorded during this study was on the order of 800 ppm.

Because of recurring channel maintenance problems in the Lower Willamette and Columbia Rivers, another sedimentation investigation program was conducted

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between July 1959 and August 1960. Sediment concentrations were found to be relatively low, as shown in earlier studies discussed above; however, annual sediment loads are significant because of the large volumes of water in the northwest streams. This is apparent from the large amounts of maintenance dredging done in the lower reaches of the Willamette and Columbia Rivers. Each year some 10 million cubic yards of material is removed from these areas. In the 1959-1960 investigations, three stations were sampled, two on the Lower Columbia River and one on the Willamette River. These three stations (Columbia River at Hood River, Oregon; Columbia River at Vancouver, Washington; and Willamette River at Newberg, Oregon) were felt to be representative of samples of sediment deposited in the problem areas in the lower reaches of the two streams. The Hood River Station was selected because it is located above Bonneville Dam in the pool area. The station at Vancouver was selected because it is representative of the Lower Columbia in the tidal reach, and the Newberg Station was chosen because of its proximity to the Lower Willamette.

Runoff from the two areas is seasonally out-of-phase -- the Columbia River runoff is basically from snowmelt and occurs in the late spring and early summer, whereas, in the Willamette River Basin, the runoff is primarily from rainfall and occurs in the late fall and winter months. Runoff magnitude for the Columbia ranges from an average of 600,000 second-feet to the largest of record 1,240,000 second-feet seldom getting lower than 80,000 as measured at The Dalles. The standard project flood for the Lower Columbia (the reach of the river below McNary Dam) is 1,550,000 cfs at The Dalles. Regulated by existing and authorized projects, it will be 850,000 cfs. The Willamette River at Portland has an average discharge of 34,000 cfs with the maximum of record being 585,000 cfs. The standard project flood is 692,000 cfs and would be 525,000 cfs regulated by existing and authorized projects.

The sampling frequency was based primarily on the availability of personnel, although samples generally were taken at approximately 2 to 4 week intervals throughout the year, depending on magnitude of flow. Results of the sampling are given on tables 2,3, and 4. A graphical representation of table 3 is shown in figure 2 as a rating curve of sediment load versus discharge.

As can be seen in the table, there are greater amounts of sediment at Vancouver than at Hood River, indicating either channel degradation or bank erosion below the project and above Vancouver; in the case of the Lower Columbia River it is both. Since sediment load is a function of stream velocity, when high flows are maintained the banks tend to erode and if low flows are maintained, sediment is deposited in the stream bed forming bars which effect channel depth and location. In addition, a constant velocity does not exist in the Lower Columbia because the flow is influenced not only by reservoir releases but also by tidal fluctuations. The head of tidal influence goes up stream as far as Bonneville tailwater, depending on the magnitude of flow. The only solution to maintain channel depth is to continue the dredging operations.

Other work in the area west of the Cascades involved the establishment of sedimentation ranges in reservoirs to monitor sediment accumulation. The general consensus, however, is that sediment is of no consequence in the area west of the Cascades, at least from a standpoint of reservoir siltation. In 1958 and again in 1966, the Portland District had an oppprtunity to substantiate this theory.

In 1958, Dorena Reservoir (located on the Row River, a tributary of the Coast Fork Willamette), was dewatered as part of a program to poison trash fish in the reservoir. The dewatering afforded the district the opportunity to make a visual reconnaissance of practically the entire reservoir, with water covering only 20 of the 1,000 acres of surface of the reservoir. The sedimentation range lines were relocated for reference at some 200 points on these ranges throughout the reservoir, vertical sections of the sediment were exposed and observed. These sections presented a clear line of demarcation between the original ground and the sediment. In the eight years since the reservoir had been filled there had been only about 0.3 feet of sedimentation deposited at most of the locations examined. The greatest depth was mid-point in the reservoir at about minimum flood control pool.

Based on this survey, it was estimated that total sediment deposits over the nine year period had been 350 acre-feet with 150 acre-feet being deposited below minimum flood control pool. The 200 acre-feet deposited in the active pool area is less than 0.3% of the active storage. On the basis of 350 acre-feet, Dorena Reservoir is silting at a rate of 39 acre-feet per year or 0.15 acre-feet per square mile of contributing drainage area per year.

In 1966 after 24 years of operation, Cottage Grove Reservoir, on the Coast Fork of the Willamette River, was dewatered for a trash fish kill. At this time, as in 1958 at Dorena, the reservoir was surveyed to find the distribution of sediment deposition over the years. In this case, 80 points were checked and it was found that silt deposits ranged from 0.2 feet to 0.6 feet. In some areas of depression and in at least one old side channel, silt was found to be 3 to 4 feet deep, but these areas were small and not at all typical of the reservoir. The following tabulation shows the number of observations along with the maximum, minimum, and average silt deposit along each range line:

COTTAGE GROVE RESERVOIR SILT DEPOSIT OBSERVATIONS - 1966 Survey

Range	Number Observations	Depth of Deposit Minimum	Maximum	Average
A	11	0.1	1.5	0.6
B	8	0.1	1.2	0.5
C	10	0.1	1.5	0.4
D	14	0.1	1.5	0.6
E	13	0.1	1.0	0.4
F	7	0.1	0.6	0.3
G	7	0.1	0.6	0.3
H	5	0.1	0.3	0.2
I	5	0.1	0.5	0.4
J-W	0 1/			
Total	80			

1/ There was very little indication of silting in the upper reach of the reservoir and no silt determinations were made between range lines "J" through "W", inclusive.

The greatest deposits were found to be located in the same relative areas as in Dorena Reservoir, which was at mid-point of the reservoir just upstream from the minimum pool elevation. The most probable reason for this pattern of disposition is the manner in which the reservoir is operated. In the winter, when the flows are high, the reservoirs are kept at a minimum pool for flood control operation and in the non-flood season they are filled. Therefore, the large inflows of sediment go directly into the inactive or dead storage area of the reservoir. Chart 1 shows a typical operation rule curve for reservoirs west of the Cascades. In addition to the sediment being deposited in the inactive pool area, the annual emptying and filling of the reservoir probably has a flushing action that tends to keep the sediment removed from the area above minimum pool. The rate of silting at Cottage Grove Reservoir was estimated to be 325 acre-feet for the 24 year period. This is about 0.13 acre-feet per square mile of drainage area, and is approximately the same as the 0.15 acre-feet per square mile per year found at Dorena. Conclusions drawn from these two studies were that sedimentation in Willamette River Reservoirs is of no consequence, particularly since 90% of the deposits occur in the area of the reservoir below the active storage. At the present time, a value of 0.15 feet per year per square mile tributary drainage area of sediment deposit is felt to be a reasonable value for all Willamette Valley Reservoirs.

Areas in Washington, west of the Cascades, are very similar to the areas in western Oregon. However, since there are few reservoirs in western Washington, there is no way at this time to prove that reservoir conditions are the same. Reservoirs currently being studied in western Washington include Wynoochee on the Wynoochee River where sediment ranges have been established, and Howard Hanson on the White River, where sedimentation ranges have been established and resurveys have been made. In both cases, the amounts of sediment have been found to be minor. At Mud Mountain Reservoir on the Green River, there does not appear to be much sediment. However, at a downstream steel lined tunnel there is evidence of deterioration that this is probably the result of a flushing action of the seasonal reservoir operation. Mud Mountain Reservoir is fed by a glacial stream and the sediment load of glacial flour is quite heavy, but the particle size is so fine that it does not settle and is not considered to be a problem. The problem here involves sediment of grain size in the range of sand. The resulting sand-blasting effect on the downstream tunnel indicates that there are large amounts of sediment transport in the river.

In the more arid areas east of the Cascades, the weather, flow, and sediment patterns are different from the west side of the Cascades. This is a high plateau area, and prior to the coming of the white man, had a good cover of various types of vegitation that largely prevented erosion and thereby eliminated most of the sediment from being carried into the streams. The introduction of agriculture, with its plowed fields and destruction of vegetative cover, caused erosion of the high ground to the valleys and into the streams. Even so, at most of the main stem points of the major streams in the area there have been no adverse effects from the sediment primarily because of the large number of projects on these streams. Each traps small amounts of sediment load. By dividing the total load among several reservoirs, no one reservoir receives a large amount, and therefore, no problems arise.

Problems do arise, however, in the smaller streams which flow into the arms of these reservoirs. The smaller streams, for the most part, flow through the agricultural areas and carry large amounts of sediment (see Figure 6) from farm lands, especially during flood periods. When these sediment ladden streams reach the reservoir areas and the velocities decrease, the suspended sediment is deposited. Since the reservoir arms are protected from wind and waves, they are ideal for recreational activities such as boating, but if they silt-up, this benefit is lost.

The Walla Walla District has recognized this problem and has been collecting data in these areas for a number of years. To illustrate the severity of the problem, some of the data that has been collected on the Walla Walla River Arm of McNary Reservoir will be discussed. McNary Reservoir is located on the Columbia just below the mouth of the Snake River. It was completed in 1953, and the local interest immediately realized the value of the Walla Walla River Arm as a boat basin and ideal location for a yacht club. The district predicted siltation but the yacht club was built anyway. The yacht club went to considerable expense to drive pile dolphins for moorage docks, build a ramp, road, and club house and construct a hoist type boat launch. Now, 17 years later, the area has been abandoned because there is less than 5 feet of water when the pool is at its lowest operating limit. A highway relocation, in order to accommodate the boat passage, was built on a high fill, and a bridge with a long center span was build to accommodate boat traffic. The bridge is no longer needed for boat passage because there is not enough depth to operate boats in this arm of the reservoir. Figures 3, 4, and 5 show cross-sections plotted on the range lines of the Walla Walla Arm for three different times. Range 3, figure 4, is the area where the yacht club was located. Examination of these cross sections show that the bottom has raised at least 5 to 6 feet and as much as 15 feet in some locations due to the sediment deposit. It can also be seen that as much as 10 feet of sediment occurred in the 1964 flood alone. This problem is prevalent throughout the area, and as yet, there appears to be no economical method of eliminating or correcting it.

The following procedures were devised by the Walla Walla District to estimate the effects of the deposits in reservoir arms.

Experience showed that sediment flowing in steep streams tends to deposit as velocities are reduced. This is the phenomena which is intended to be simulated by these procedures.

a. Check the average annual sediment contribution in the stream for potential volume accumulation.

b. Where the tributary stream enters the main stream, assume the sediment edge will lie on the flat slope of 1:20 back from the edge of the precut stream if there is sufficient sediment accumulation to fill this.

c. Assume that sediment will deposit at this outer edge to the reservoir surface elevation and will lie in a fairly plain level with the tributary arm of the reservoir.

d. Assume that in this flat plain there will exist a channel able to carry a 2 year flood flow within bank of a 5 year flood flow with over flow and a mean sectional channel velocity which is neither silting nor eroding (say 2-3 feet per second). Compute backwater up the tributary stream assuming the flood plain along the channel maintains the same slope as the water surface.

e. For determination of the changed extent of backwater, compute backwater from main reservoir full-pool elevation up the channel by adjoining flood plain until it intersects backwater elevations without sediment deposits in the reservoir. Assume that the occurrences of the flood used will not erode the channel or the flood plain. Use a reasonable design flood such as a 50 or 100 year flood or SPF. Use end of backwater as limit of level to be acquired for the reservoir. The above approach is thought to be a reasonable way of estimating sediment accumulation in reservoir arms.

Another problem found in the Walla Walla area is channel degradation. Although not strictly a sediment problem, it is sufficiently important to be discussed at this time. It involves the Walla Walla River at Milton Freewater, Oregon. The channel of the Walla Walla River near Milton Freewater is an improved channel constructed by the Corps in 1948-1950. According to the Definate Project Report the improved channel was trapesoidal in shape, and had a bottom width of 120 to 200 feet. Design discharge was 18,600 cfs. Slope of the channel varied between .01 and .015, and design depth was in the order of 10 feet. Banks were levees with a slope of 1 on 2 and riprapped. Riprap was extended below the channel bed 4 feet and bed material was backfilled over this lower extension of the riprap.

A hydrologist from the Walla Walla visited this channel on a number of occasions subsequent to 1949. He noted that much of the backfill over riprap extending below the channel bed had washed out, that tires and other debris had lodged in mid-channel, and that sediments had accumulated around these lodged obstacles. Hence, relatively low flows of perhaps 100 cfs, were generally concentrated along the edge of one or both of the levees and in many places islands extended in mid-stream. During one visit it was noted that in a few places where the current had been severe adjacent to the bank, the bed had washed out deep enough so that some of the riprap had sloughed off.

It is possible that the channel deterioration and sloughing off of the riprap caused streamflows to concentrate near the levees and to wander back and forth across the channel, establishing a meandering pattern and forming bands which directed the flows increasingly transverse to the axis of the flow line. Finally, during high flows, an almost harmonic meandering back and forth across the channel occurred with high velocities being directed toward the stream banks. Pictures of the results of this action are available in the 1964 Post Flood Report by the Walla Walla District.

Seattle District has begun a study that is unique in our area. The district is establishing sedimentation ranges in a reach of river downstream from Libby Reservoir. This is a problem area where the steep stream flows into a flat area and presents problems of channel agradation and flooding.

Walla Walla District has noted the same type problem in the past, but this is the first time than an actual study on the subject has been undertaken.

Alaska District has not conducted sedimentation studies or data collection programs on the rivers in their district. Observation reveals that in certain areas, the streams are clear and seem to carry very little sediment load. Other streams, those fed by glaciers, are quite murkey due to the heavy concentrations of glacial flour. The Alaska District will probably begin sedimentation programs in connection with the Chena River Flood Control Project which has been approved. Work on this project will probably be done within the next two or three years.

TABLE 1

Sediment Concentration and Load 3/

Station	Stream	Drainage area, square miles	Number samples obtained	Sediment Concentration in p.p.m. Max.	Min.	Total sediment load in tons Water years 1949	1950	1951 1/
Farmington	Tualatin R.	568	320	390	10	123,030	60,200	56,220
Canby	Molalla R.	323	313	560	T	95,620	73,370	56,940
Aurora	Pudding R.	493	330	315	T	96,270	60,780	66,950
Whiteson	S. Yamhill R.	502	342	800	12	280,480	112,380	134,180
Salem	Willamette R.	7,280	327	400	10	1,926,250	1,738,900	1,916,960
Suver	Luckiamute R.	236	342	410	T	90,700	46,240	49,150
Jefferson	Santiam R.	1,790	327	508	T	574,850	622,800	633,340
Mahama	N. Santiam R.	110	252	250	T	148,910	110,630	130,750
Waterloo	S. Santiam R.	640	321	370	T	212,420	178,090	187,140
Albany	Calapooya R.	362	323	340	10	90,340	60,590	54,590
Philomath	Marys R.	155	339	500	T	65,700	28,080	46,390
Bel. Fern Ridge Dam	Long Tom R.	252	328	279	10	43,410	45,230	57,580
Elmira	Long Tom R.	100	324	126	7	12,640	9,570	12,500
Elmira (nr.)	Coyote Cr.	100	330	156	16	9,890	10,820	16,050
Coburg	McKenzie R.	1,310	331	240	T	255,310	105,630	233,010
Springfield	Willamette R.	2,030	318	350	T	380,000	276,000	614,900
London	Coast Fork Will. R.	69	328	400	T	13,000	7,650	13,640
Bel. Dam	Coast Fork Will. R.	108	318	260	T	8,890	9,420	15,110
Star	Row R.	211	331	330	T	27,150	12,550	8,490
Bel. Dam	Row R.	265	108	130	T	-	1,360 2/	32,780
Lowell	Middle Fork Will. R.	994	312	380	T	93,660	63,210	390,990
Vancouver	Columbia R.	241,000	394	420	12	-	-	-

1/ Partial year, Oct - Jun
2/ Partial year, May - Sep
3/ Results of Dec 1948 - Jul 1951 Sedimentation Investigation

TABLE 2

COLUMBIA RIVER AT HOOD RIVER

Q - 1000 cfs	Sediment Discharge in 1000 tons/day 1/			
	Falling Stage		Rising Stage	
	1000 tons per day	cubic yards per day	1000 tons per day	cubic yards per day
100	5	4,630	9	8,330
200	14	12,960	24	22,220
300	27	25,000	44	40,740
400	44	40,740	66	61,110
500	65	60,190	93	86,110
600	88	81,480	122	112,960
700	115	106,480	155	143,520

1/ Results of July 1959 - August 1960 Sedimentation Investigation

TABLE 3

COLUMBIA RIVER AT VANCOUVER

Q - 1000 cfs	Sediment Discharge in 1000 tons/day[1] Falling Stage 1000 tons per day	Falling Stage cubic yards per day	Rising Stage 1000 tons per day	Rising Stage cubic yards per day
150	14	12,960	38	35,190
200	24	22,220	54	50,000
300	52	48,150	98	90,740
400	96	88,890	158	146,300
500	165	152,780	250	231,480
600	280	259,260	400	370,370
700	450	416,660	600	555,550
800	700	648,140	920	851,850
900	1100	1,018,500	1310	1,212,960

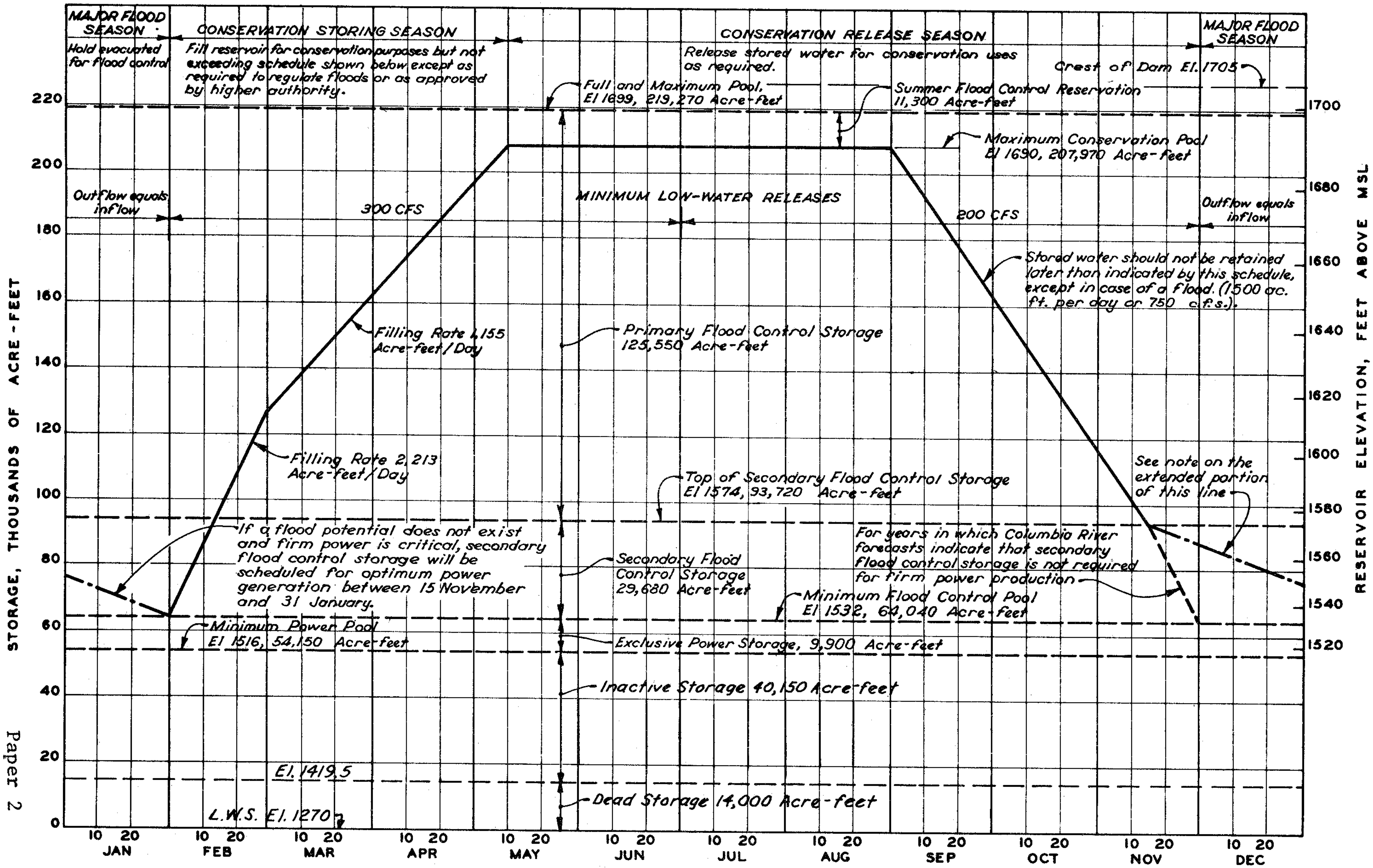
1/ Results of July 1959 - August 1960 Sedimentation Investigation

TABLE 4

WILLAMETTE RIVER AT NEWBERG

Q - 1000 cfs	Sediment Discharge in tons/day 1/			
	Falling Stage		Rising Stage	
	tons per day	cubic yards per day	tons per day	cubic yards per day
6	410	380	580	537
8	510	472	830	768
10	610	565	1,100	1,020
20	1,250	1,160	3,200	2,960
30	2,200	2,040	6,200	5,740
40	3,500	3,240	10,300	9,540
60	7,000	6,480	22,000	20,370
80	12,000	11,110	37,000	34,260
100	20,000	18,520	56,000	51,850
120	29,000	26,850	78,000	72,220

1/ Results of July 1959 - August 1960 Sedimentation Investigation



COUGAR RESERVOIR
SEASONAL REGULATION SCHEDULE

CHART NO. 1

WATER RESOURCE DEVELOPMENT
NORTH PACIFIC DIVISION

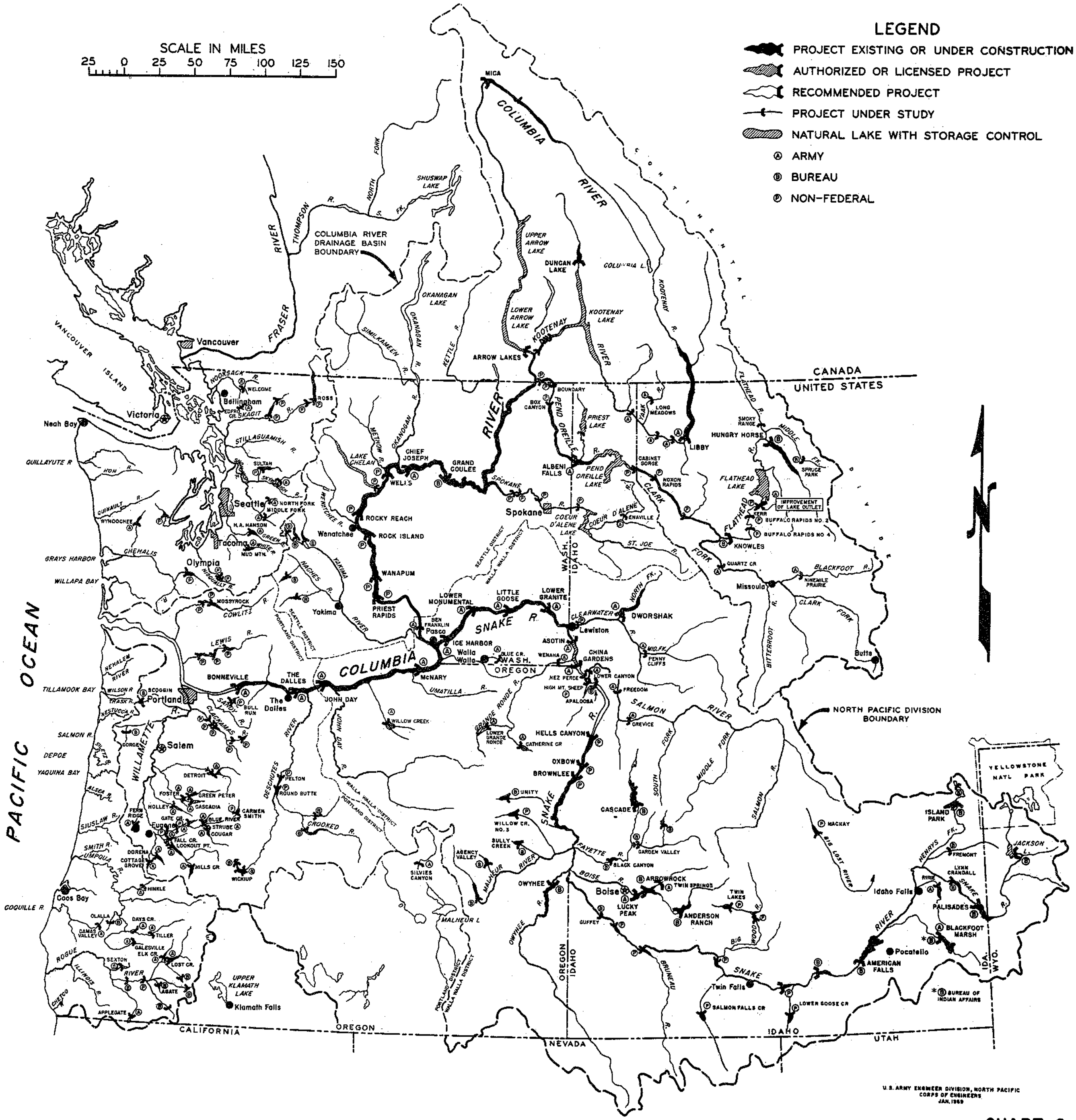
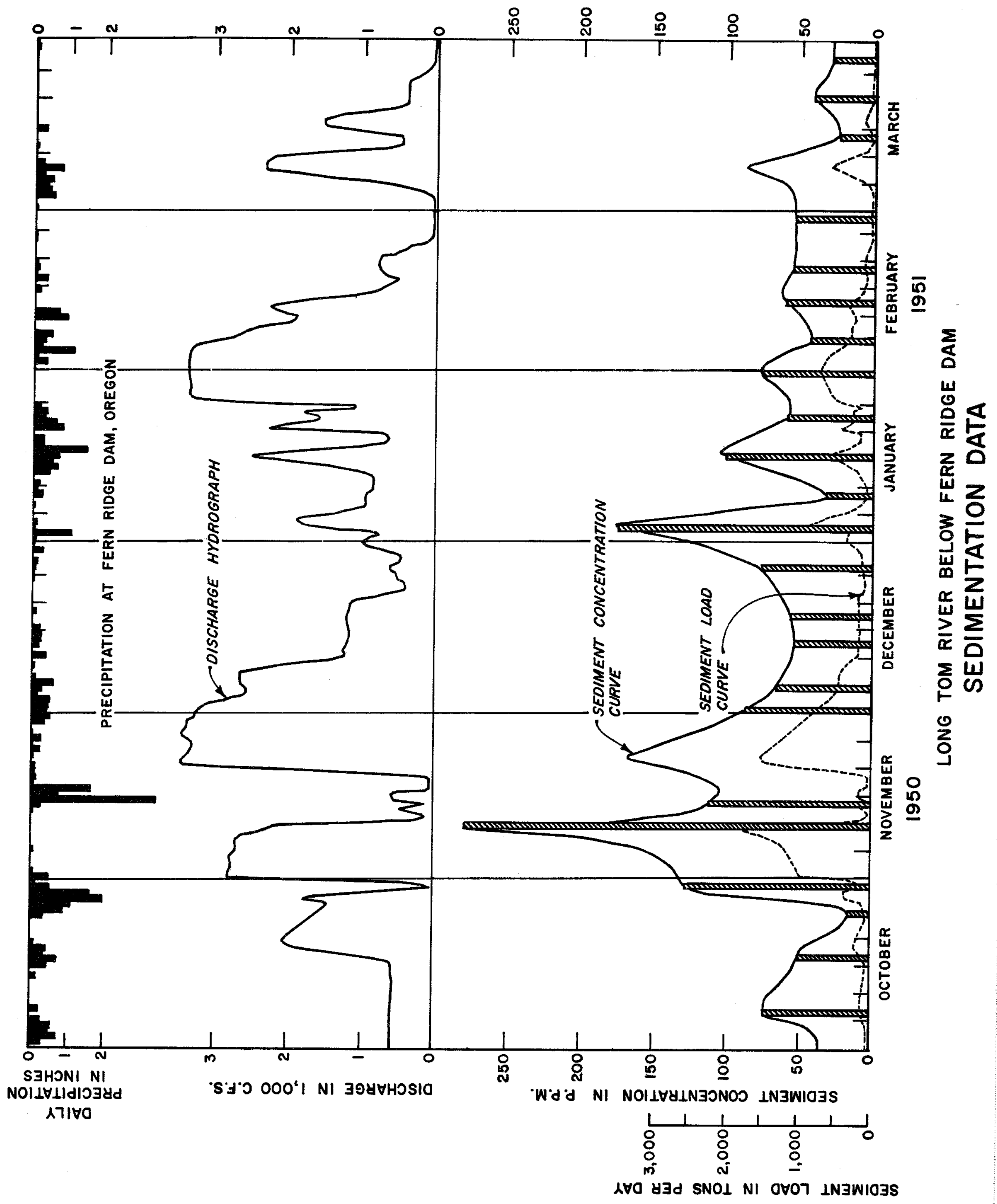
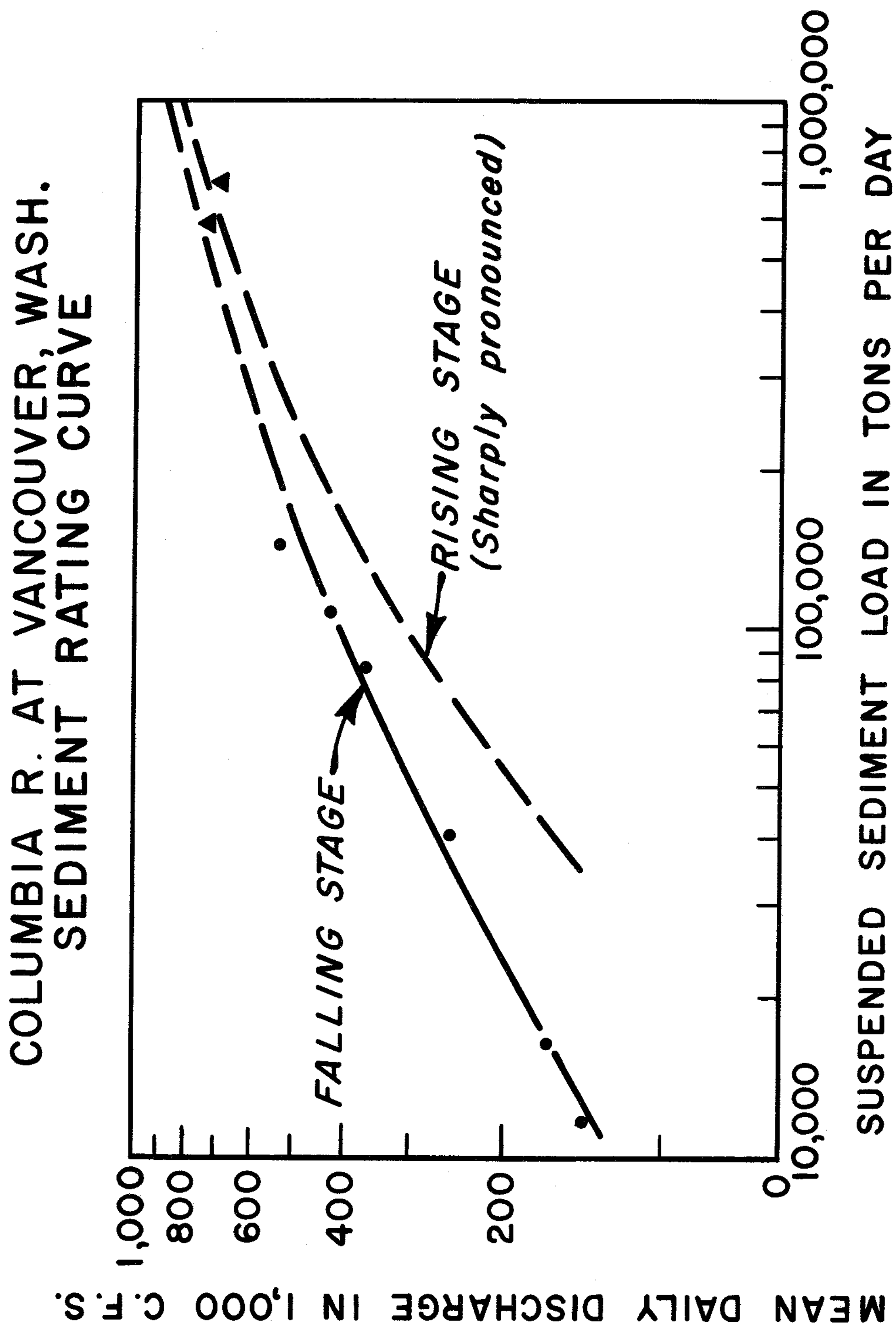


CHART 2



LONG TOM RIVER BELOW FERN RIDGE DAM
SEDIMENTATION DATA
PRECIPITATION AT FERN RIDGE DAM, OREGON
DISCHARGE HYDROGRAPH
SEDIMENT CONCENTRATION CURVE
SEDIMENT LOAD CURVE
DAILY PRECIPITATION IN INCHES
DISCHARGE IN 1,000 C.F.S.
SEDIMENT CONCENTRATION IN P.P.M.
SEDIMENT LOAD IN TONS PER DAY
OCTOBER
NOVEMBER
1950
DECEMBER
JANUARY
FEBRUARY
1951
MARCH

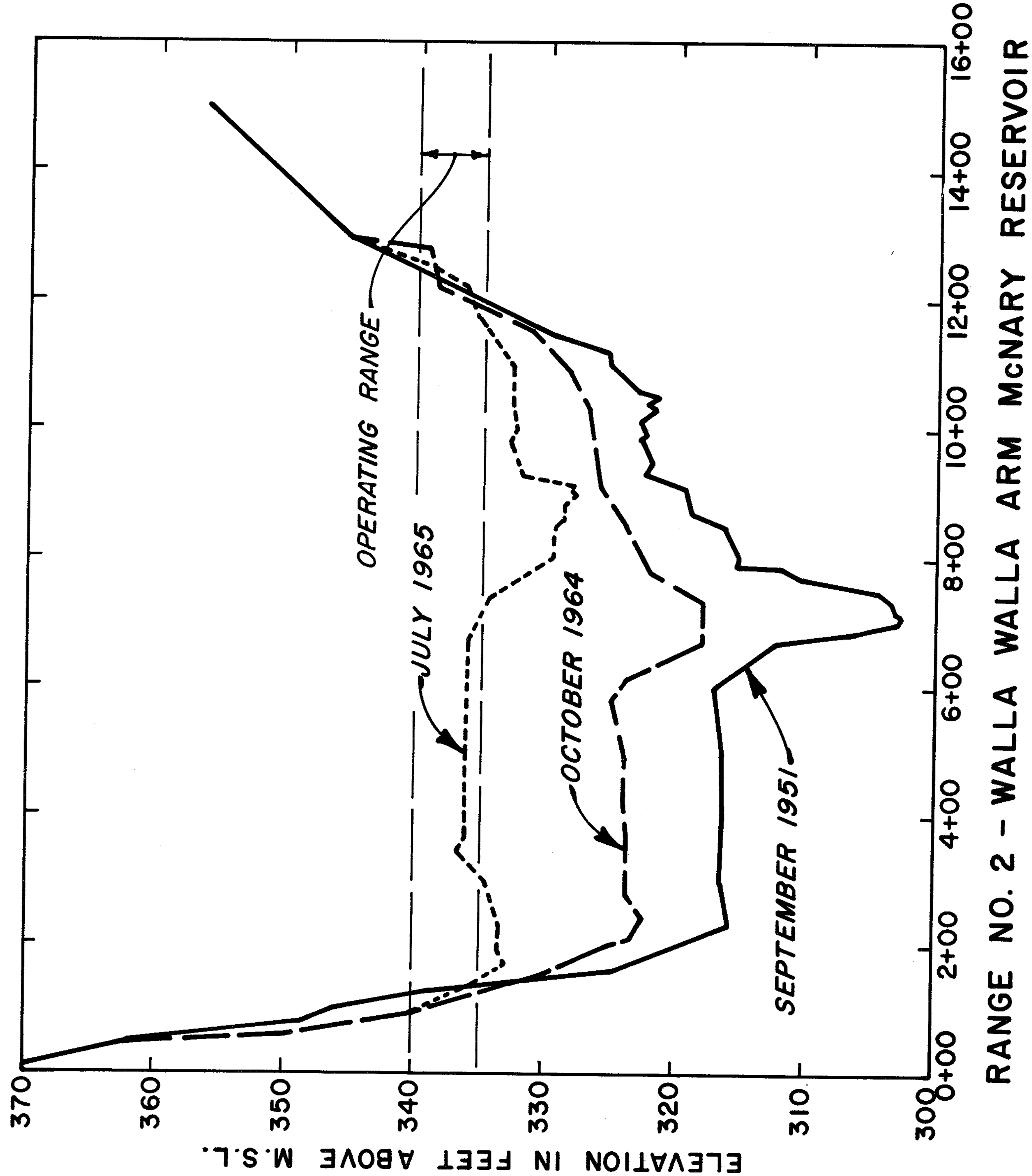
FIG. 1



COLUMBIA R. AT VANCOUVER, WASH.
SEDIMENT RATING CURVE
RISING STAGE
(Sharply pronounced)
FALLING STAGE
MEAN DAILY DISCHARGE IN 1,000 C.F.S.
1,000
800
600
400
200
0
10,000
100,000
1,000,000
SUSPENDED SEDIMENT LOAD IN TONS PER DAY

FIG. 2

Paper 2



RANGE NO. 2 - WALLA WALLA ARM McNARY RESERVOIR
ELEVATION IN FEET ABOVE M.S.L.
370
360
350
340
330
320
310
300
0+00
2+00
4+00
6+00
8+00
10+00
12+00
14+00
16+00
OPERATING RANGE
JULY 1965
OCTOBER 1964
SEPTEMBER 1951

FIG. 3

Paper 2

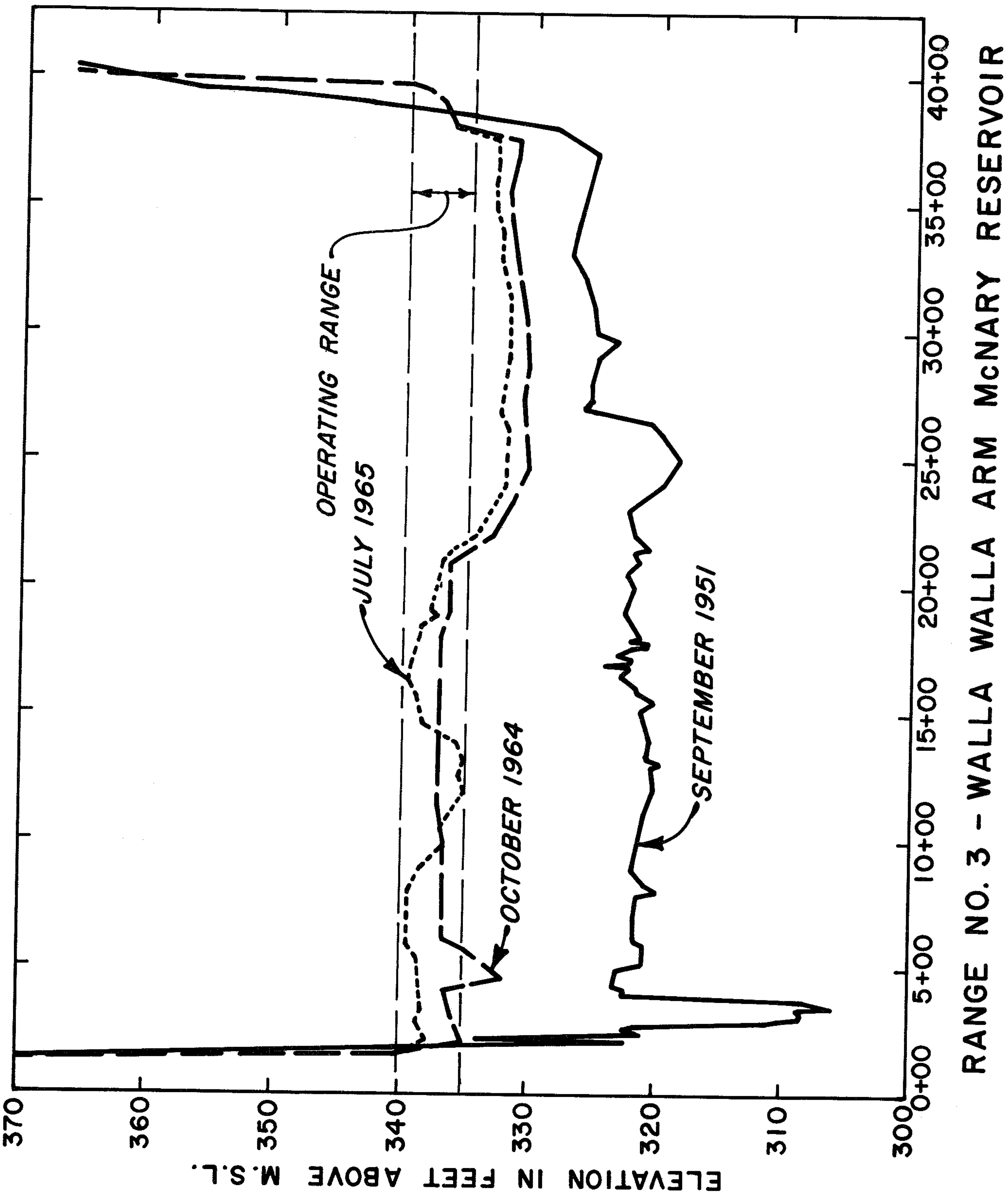

RANGE NO. 3 - WALLA WALLA ARM McNARY RESERVOIR
ELEVATION IN FEET ABOVE M.S.L.
370
360
350
340
330
320
310
300
0+00
5+00
10+00
15+00
20+00
25+00
30+00
35+00
40+00
OPERATING RANGE
JULY 1965
OCTOBER 1964
SEPTEMBER 1951

FIG. 4

Paper 2

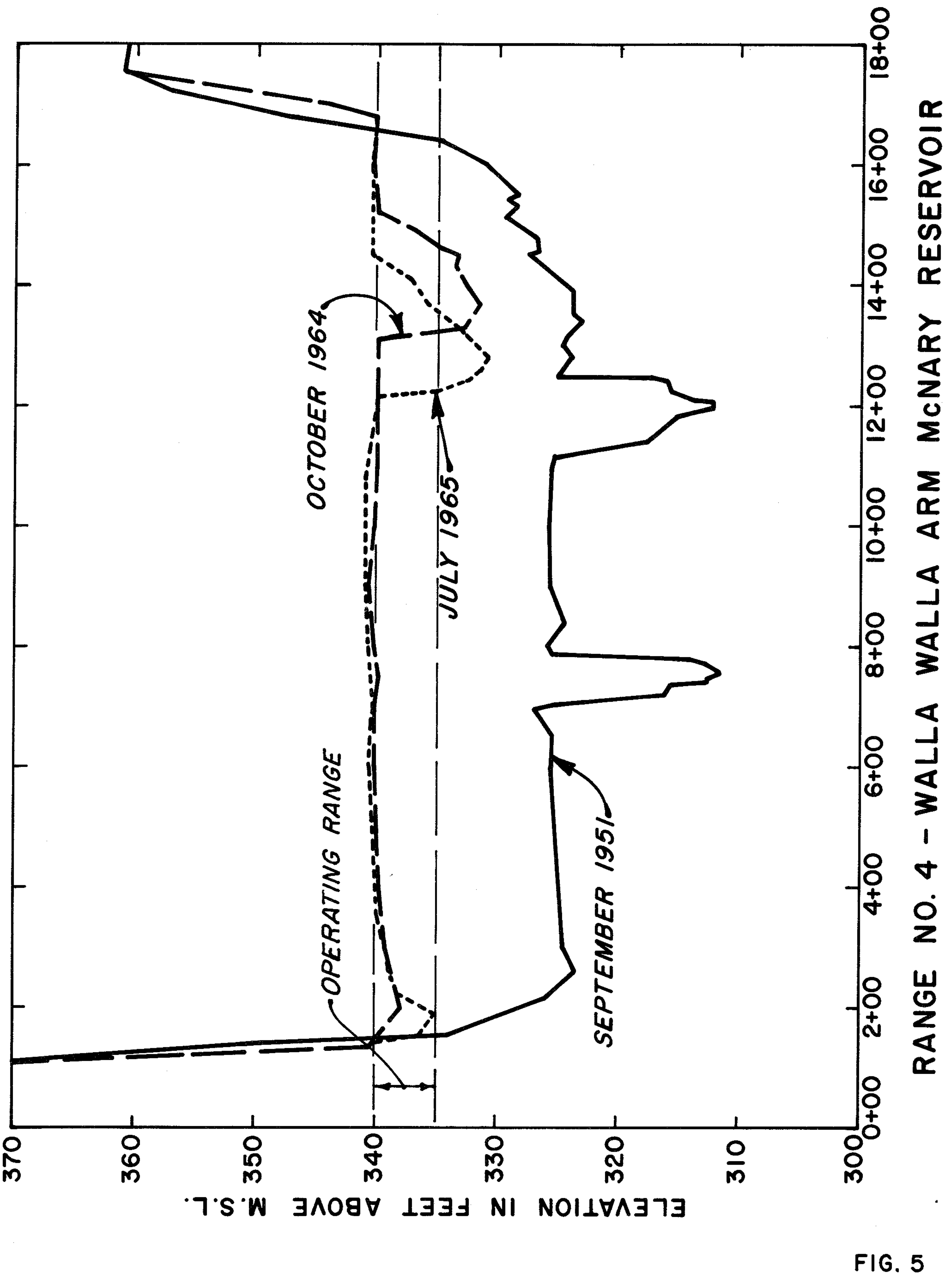
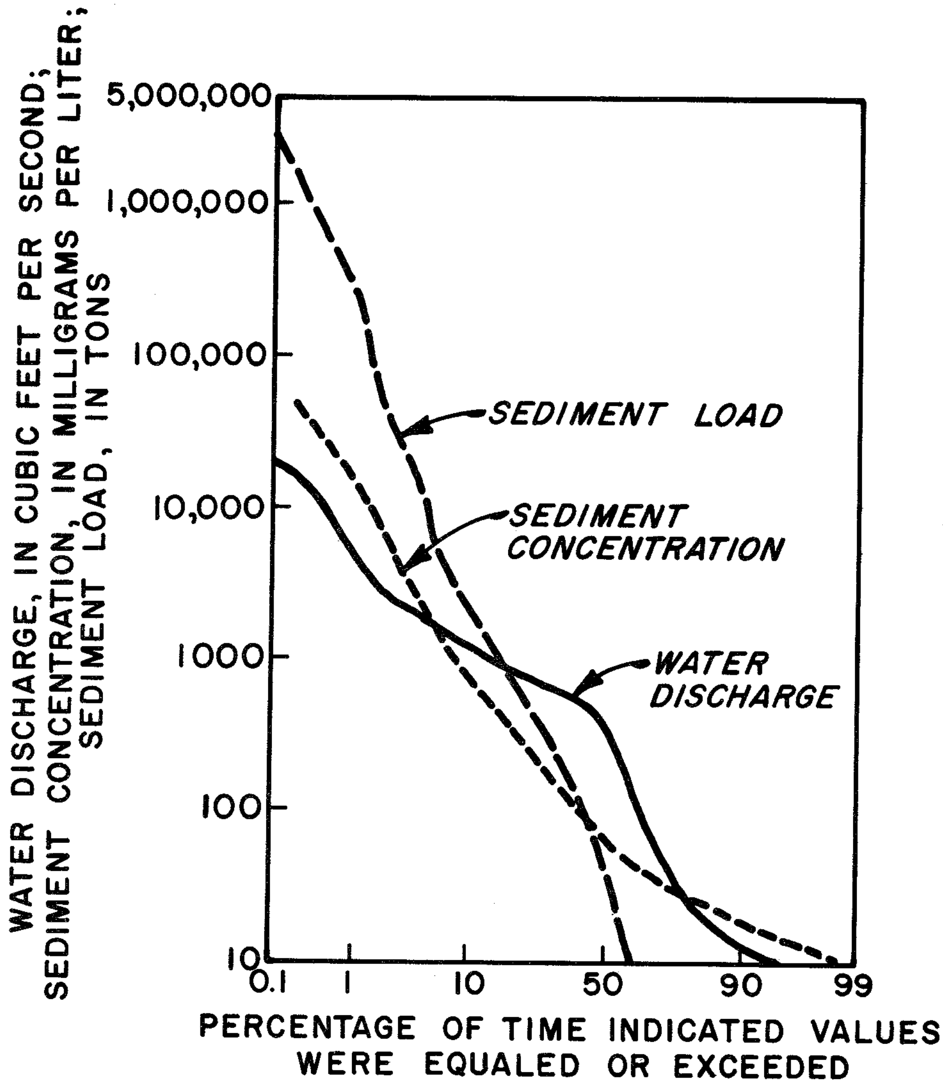

RANGE NO. 4 - WALLA WALLA ARM McNARY RESERVOIR
ELEVATION IN FEET ABOVE M.S.L.
370
360
350
340
330
320
310
300
0+00
2+00
4+00
6+00
8+00
10+00
12+00
14+00
16+00
18+00
OPERATING RANGE
OCTOBER 1964
JULY 1965
SEPTEMBER 1951

FIG. 5

Paper 2



FREQUENCY CURVES OF MEAN DAILY WATER DISCHARGE, SEDIMENT CONCENTRATION, AND SEDIMENT LOAD, WALLA WALLA RIVER NEAR TOUCHET, JULY 1962 - JUNE 1965

FIG. 6

AN EXAMINATION OF ENGINEERING PRACTICES RELATING TO RESERVOIR SEDIMENT INVESTIGATIONS

by

Brice L. Hobbs[1]

INTRODUCTION

The discussions in this paper are intended to serve as a stimulus for thought regarding present practices in sediment engineering investigations relating to the planning, design and operation of dams and reservoirs controlled by the Corps of Engineers and the need to improve capabilities in these areas. Certain views are presented regarding the adequacy of available basic data to meet specific requirements for engineering studies and some suggestions are offered for improving the overall program of reservoir sediment investigations conducted by the Corps of Engineers. It is recognized that the Corps has been responsible for much fine work in investigations of reservoir sedimentation and that the results of these studies represent the foundation for future studies that would place the Corps in a position of leadership in sediment engineering relating to large reservoirs. Accordingly, the attempt to focus attention on weaknesses in the Corps practices is deliberate and meant to be constructive.

Reservoir sedimentation processes represent complex and ever changing combinations of a large number of factors which are also variable. Some of the more important factors are discussed briefly in literature references. 1/ 2/ Except to make certain illustrations, the factors involved in reservoir sedimentation and how it is estimated will not be discussed in this paper.

RESERVOIR SEDIMENT PROBLEMS

Reservoir sediment problems cannot be divorced from the contributing factors such as the stability of soils and channels and sediment transportation. Accordingly, sediment problems in a reservoir should, insofar as practicable, be considered in the context of a long-range comprehensive basin plan for water resources development. Reservoir sediment problems fall into one of the following general groups: (1) depletion of storage space by deliberate entrapment or unavoidable entrapment resulting from operation for other project purposes; (2) aggradation of channels tributary to the reservoir; (3) delta problems; (4) shore erosion problems; and (5) problems associated with reservoir influences below dams. The following extract, from a paper distributed to Corps installations with Engineer Technical Letter No. 1110-2-64, 2/ is repeated here for convenience of reference.

[1] Formerly Hydraulic Engineer, Office Chief of Engineers

Depletion of Storage Space - If volumetric reductions of reservoir storage space allocated for various purposes represented the only problems associated with reservoir sedimentation, forecast information of fractional distributions of total deposits would not serve any particularly useful purpose even where rapid gross depletion is anticipated. If such were the case it would be necessary only to make appropriate reallocations as would be indicated by periodic resurveys. However, the significance of storage depletion and other related problems depend generally upon the average sedimentation rates and progressive distribution of deposits. On alluvial streams, it is usually important to have forecast estimates of the probable distributions of deposits both with respect to areal location and volumetric accumulations in various elevation zones. Such information is useful in connection with planning and design considerations to assure that serious encroachments upon space allocated for purposes other than sediment retention will not occur during the period used for economic analysis of the project. Some important sediment distribution problems are discussed further in the following subparagraphs.

Aggradation of Tributary Channels - A reservoir on an alluvial stream is one of the more important man-made influences which may affect channel conditions. The aggradation of channels which sometimes occurs above reservoirs is an extension of the reservoir sedimentation processes which may adversely affect drainage conditions and aggravate flooding problems on adjacent lands. Relatively small fractions of the total accumulations are usually involved in the aggradation of channels in the reaches of reservoir-backwater influences above established pool levels and future dimensions of aggraded channels cannot be accurately forecast by known methods.

Aesthetic Effects - Regardless of the need for sediment distribution estimates for other purposes, it is occasionally important to foresee future conditions which might be unsightly and therefore objectionable to people residing nearby.

Depletion of Storage Space in Single-Purpose Reservoirs - Normally a reasonable estimate of the total volume of sediment anticipated during the period used for economic considerations is all that is necessary for establishing storage requirements in single-purpose reservoirs, and advance information regarding the locations of the deposits is usually not needed. Exceptions may be found in cases where substantial inactive storage is required in reservoirs operated primarily for power production.

Depletion of Storage Space in Multiple-Purpose Reservoirs - In cases where sediment yields are appreciable, advance information of probable future distributions of sediment deposits is important in connection with planning and design considerations of storage depletion regardless of the project purposes. Misjudgments involved in the initial allocations of storage space cannot always be satisfactorily rectified by reallocations of space remaining at some future date. For example, head limitations might preclude lowering the elevation of the minimum power pool.

Depletion of Space Where Water is Stored for Recreational Purposes - Recently, there has been a rapid increase in demands for storage of water for recreational activities in artificial lakes. The needs are usually satisfied by: use of water stored primarily for other purposes; provisions for perpetual storage of a given volume of water regardless of pool elevation; specific allocations of storage below a given pool elevation; or arrangement for regulation so as to provide for a minimum pool having storage not exceeding that provided for conservation and the undepleted space initially reserved for sediment. There is general agreement regarding the importance of recreational needs; therefore, the problems that may be expected to result from unfavorable sediment distributions should be recognized. For example, a plan to continuously provide a small pool of fixed volume in the lowest elevation zone of remaining space may become completely unsatisfactory for the planned activity relatively early in the life of the project. Also, decisions are often made, after completion of the design stage and without benefit of additional engineering study, to regulate a reservoir so as to utilize space reserved for sediment deposits for recreational or other conservation purposes. In such cases there is no opportunity for changing the total storage, therefore, the effects of the change on sediment distribution expected to result from the change in regulation procedures should be carefully examined.

Shore Erosion - Shore erosion and bank caving processes frequently create beach and boat harbor problems. Movement of material by these processes may cause an exchange of storage space between elevation-zones or a net storage loss or both.

Utilization of Delta and Backswamp Areas - Interests opposing the construction of reservoirs frequently cite sedimentation as one of the horrible consequences of these developments and the general public is led to believe that the results are always entirely bad. Actually, the program for wildlife propogation, by the U.S. Fish and Wildlife Service, in the delta areas of Denison Reservoir is reported to be quite successful. This represents a type of planning problem that has not had proper consideration in the past.

A few of the adjunct problems peculiar to individual projects which have been encountered are: (1) isolation of shore developments by advancing deltas formations; (2) increased water loss problems by evaporation and transpiration; (3) drainage problems associated with ground-water changes; (4) aggravation of pollution problems; (5) removal of sediment deposits by dredging and sluicing; and (6) discharging materials transported by density currents.

BRIEF HISTORY OF RESERVOIR SEDIMENT MEASUREMENT AND DOCUMENTATION OF RECORDS IN U.S.

An important impetus was given to reservoir sediment investigations by the survey program begun by the U.S. Soil Conservation Service in the mid 1930's. At that time the Corps of Engineers had few reservoirs in operation and it was decided that provisions should be made for sediment surveys of all reservoirs controlled by the Corps.

There are over twelve hundred reservoirs in the United States on which records of sediment measurements have been published. 3/ These reservoirs range in size from small farm ponds to large multiple purpose projects having capacities ranging up to about 30,000,000 acre feet. The kinds of sediment data collected vary from simple records of sediment quantities dredged from debris basins to detailed information obtained by comprehensive surveys. In several cases measurements and observations are made to obtain related data on sediment sources, water-sediment inflows, quantities of sediment discharged through outlets, deposit densities, physical characteristics of materials and distributions of deposits. Also, there exists much pertinent hydrologic and hydraulic data concerning the regulation of reservoirs, but such records have usually been collected with little or no thought given to documentation for use in sedimentation studies.

The reservoir sediment investigation programs conducted by the U.S. Bureau of Reclamation and the Tennessee Valley Authority are similar to those of the Corps. It should be mentioned here that the sediment survey investigations of Lake Mead, are probably the most comprehensive of any conducted on a single large reservoir.

All agencies of the U.S. Government having responsibilities for reservoir sediment investigations are represented on the Committee on Sedimentation of the U.S. Water Resources Council. In the interest of standardization, this committee prepared a form for summarizing pertinent data on dams and reservoirs, contributing drainage areas and related information on capacity changes. All U.S. agencies having responsibilities for reservoir surveys have adopted this form and the instructions for completing the sediment summaries which are assembled and published periodically under the auspices of the Committee on Sedimentation.

ORGANIZATIONAL ARRANGEMENTS FOR SEDIMENT INVESTIGATIONS CONDUCTED BY THE CORPS OF ENGINEERS

The extent of sediment investigations performed by the Corps varies considerably. The activities vary from those in one or two offices requiring the full-time services of one or more engineers to those performed in several offices where the problems are considered minor and the limited sediment studies are incidental to engineering studies made for other purposes. Organizational arrangements for conducting sediment studies also vary considerably among offices. A review made in 1969 indicated that only the offices of the Missouri River Division and the Omaha District have sediment sections. Organization charts for virtually all other Divisions and District Offices show little or nothing regarding assignment of responsibility for sediment studies. Reports and information obtained informally indicate that sediment engineering studies are assigned variously to the following branches and sections of Engineering Divisions: Survey; Hydrology; River Stabilization; Planning; Coastal and Estaury and Delta Sections. Other organizational units responsible for studies relating to sediment engineering include those concerned with geology, potomology, soils, channel improvement, subsurface exploration, and hydrography.

RESERVOIR SURVEY PRACTICES

Prior to 1961, the Corps of Engineers had no published instructions for conducting reservoir sediment investigations. The frequency of surveys, the kinds of data collected, and the extent of analyses made were determined by somewhat arbitrary decisions based upon a judgment of the importance of the problems. This is still true to some degree as indicated by report formats which have varied not only with the seriousness of the sediment problems, but from office to office. Some instructions which evolved from the early practices are contained in Engineering Manual 1110-2-4000. 4/ This manual provides information and instructions for use in planning and conducting programs for reservoir sedimentation investigations by the Corps of Engineers . In general, all instructions in this manual are still valid, but as indicated in the following discussions, a review of the experience gained during the past eight years should provide a sound basis for preparing a revised edition that contains additional and more specific instructions.

As might be expected, considerable diversity in survey practices in different regions of the United States still exists, and this applies to the types of surveys as well as the sophistication of equipment and techniques employed. Extremes of organizational arrangement for conducting surveys is represented by one District office which has two field parties, employed full time on sediment surveys, and another which has a contract arrangement for reservoir surveys to be conducted by another agency of the U.S. Government. Much of this diversity reflects variations in seriousness of the sediment problems, survey objectives, and the ingenuity of the engineers. However, some of the variety seems to be the result of individual opinions and changes in points of view which are not explained. For example, there are still no clearly defined guide lines regarding the scheduling of reservoir surveys. Early in the programs some reservoirs, such as John Martin on the Arkansas River, were surveyed annually, but, in recent years, the intervals between surveys have been considerably longer. Reasons for the decrease in the frequency of resurveys are obscure.

In general, complete resurveys of most reservoirs controlled by the Corps are initially scheduled to be repeated at intervals of 5 or 10 years, or soon after the occurance of a rare flood event. In some cases, programs presented for approval provide for resurveys to be scheduled on the basis of need indicated by reconnaissance or partial resurveys of selected ranges, but there is usually no specific indication of what the signs of need are expected to be.

In many cases, scheduled resurveys are delayed one or more years for lack of funds and other reasons. This suggests that the program objectives were not realistic or that an insufficient number of resurveys are being made to conform with the objectives of the investigation. Still, it is a rare case when someone expresses concern about the urgency of the delayed investigation.

Some time ago a representative of the New England Division called attention to the fact that measured sediment accumulations in several reservoirs in that region are insignificant and recommended discontinuance of the practice of installing facilities for measuring sediment in future projects in that region. It was finally decided that it is important to be in position to have the results of actual measurements, in order to refute unfounded accusations and claims regarding sediment deposition and that at least a few sediment index ranges and/or markers for measuring depths of deposits should be installed at selected locations in all reservoirs. Perhaps it is again time to review available records and reconsider the previous position.

NEED FOR NEW AND IMPROVED SEDIMENT ENGINEERING TOOLS

Sometimes the sediment engineer suspects that others responsible for storage allocations have little appreciation for the implications regarding the requirements for measuring the depletion of storage space. Trinidad Reservoir, a multiple purpose project on the Purgatorie River in Colorado, is a good example. One plan of space allocation, believed to be the one finally adopted, provides for regulation so that a relatively small volume of water will remain for recreation purposes after all water stored for flood control and irrigation has been released. In view of the large and variable amounts of sediment transported by the stream, one can imagine how closely the reservoir will have to be monitored by some method to determine how much flood water will need be released following each significant runoff event. Also, consider the probable conflicts between irrigation and recreation interests if the space depleted by sediment is under estimated at the onset of a dry period. If there is a simple inexpensive and relatively accurate method for continuous accounting of space remaining, it would seem that engineers responsible for regulation of releases from Trinidad and other similar projects would certainly be interested. If such a method is not available, it is conceivable that complete evacuation of the flood control pool will need to be delayed until a survey is completed.

The engineers responsible for preparing reports often find they must resort to conjectural statements regarding ungaged areas, unmeasured bed-material loads and consolidation of deposits in order to explain differences between quantities measured by surveys and those determined by sediment sampling and water discharge measurements. On this point the question immediately arises as to the wisdom of continuing the suspended sediment records in cases where there are significant differences and the primary objective is to periodically determine depleted storage space for operation purposes. The same is true of comparisons of values estimated for planning and design purposes with quantities determined by surveys. When questions are posed on the latter point, there seems always to be a considerable degree of speculation in the answers, which usually contain elements of the same stock explanations. If measured values differ it is usually contended that hydrologic conditions and hence the sediment delivery rates during the period in question were not normal. It seems reasonable that the application of statistical methods would improve engineering capability in this area.

The lack of knowledge in several other areas of sediment engineering, makes necessary a reliance on assumptions that would be unacceptable in other engineering fields. For example, it has not been demonstrated that any completely reliable methods for estimating future distributions of reservoir sediment deposits have been developed.

ANALYSIS, EVALUATION AND UTILIZATION OF EXISTING RECORDS

There exists a great mass of sedimentation data and related hydrologic data published and in the files of Corps of Engineers installations. The writer believes it is fair to say that only a small fraction of the full potential value of these records has been realized, and that this is attributable, in part, to the inordinate lag of the analysis and evaluation studies behind the data collection programs, which in turn seems largely attributable to passive attitudes of engineers concerned with other problems that demand more immediate attention.

Many of the plans adopted for the early programs contained only vague explanations of the reasons for proposals to collect certain data. Some have continued for more than 20 years in spite of the fact that no meaningful correlations were developed between sediment discharge records and information obtained from one or two resurveys, the last of which was conducted over 10 years ago. Seeming indifference is further manifest by numerous report endorsements containing no comments.

With some exceptions there are signs that most basic reservoir sedimentation data collected are not being used to the best advantage. Also, there is evidence that some data are being collected that are not particularly helpful, and that some highly desirable information is being overlooked or neglected. For example, very few of the past reports on reservoir sediment investigations contain definitive information on sediment particle sizes found in deposits or pool-elevation durations and corresponding conditions of inflow.

There is a general need for improved criteria for collecting basic sediment discharge data to serve various purposes. Also, there is a need in some offices for improvement in practices of compiling, reviewing, and publication of sediment discharge records. Often the records of annual values or average annual values are the only ones published. These appear only in project planning and design reports, while the details of daily and monthly values and important information regarding the station and drainage basin become buried in the files.

In particular, the records of suspended sediment discharge at reservoir inflow stations should be reviewed and analyzed to arrive at conclusive answers to the following questions: should the existing program of sediment sampling be continued without modification? Are additional stations needed to serve the objectives of sediment sampling program? Should operation of some or all existing stations be discontinued? Are estimates of deposition based upon the suspended sediment records and estimates of unmeasured sediment more reliable than the single points of correlations represented by records of reservoir surveys? In this connection there is a need to develop general guide lines regarding desirable periods of record and limits of confidence to be placed in estimates of long period sediment yields and deliveries determined from short-period sediment discharge records.

Any review of the reservoir sediment investigations program should include an examination of the adequacy of geographic coverage represented by reservoirs where programs have been established to meet planning and design objectives.

SOME CONCLUSIONS AND SUGGESTIONS FOR FUTURE INVESTIGATIONS

The evidence indicates an urgent need for a general re-examination of existing programs for reservoir sediment investigation in considering both the need for such records and how they will be used. However, it would be unwise to attempt to improve the programs by making many immediate modifications. Present knowledge and the limitations of funds and personnel make this an unrealistic approach. Nevertheless, it is the writer's conviction that economies could be realized and that the slow evolution of improved sediment engineering capabilities would be greatly accelerated by a thorough analysis and evaluation of available reservoir sedimentation data and related hydrologic information. It is envisioned that such a study would have the following objectives: (1) to consider the adequacy of the regional coverage of project programs having planning and design objectives; (2) to develop improved criteria for determining requirements for reservoir survey measurements and related observations; (3) to improve the quality and utility of reports on sedimentation surveys; and (4) to develop improved sediment engineering methods and procedures.

Such an undertaking would require the cooperation of A-1 field installations having control of reservoirs. It would also be essential for the study to be conducted by personnel under the administrative control of a single installation. Two qualified engineers, having expert knowledge of computer techniques and access to the equipment, probably could complete the study relating to objectives (1) through (5) in less than two years. At the end of that time these men should be in a position to chart a clear course for development of some new and improved sediment engineering methods.

No estimate of the total cost of the suggested investigation has been made, but it seems reasonable that the cost of the study could and should be funded by diverting available operation and maintenance monies saved by reasonable curtailment of the reservoir sediment investigation program. This can be accomplished by selective substitution of partial resurveys for certain complete resurveys that are scheduled and by delaying some scheduled resurveys.

The Corps now has in the order of 450 reservoirs complete, under construction, and authorized. This is exclusive of many single purpose projects for navigation and debris control for which sediment investigation programs are planned or in progress. Available information indicates that through 1965 the Corps had completed one or more resurveys on about 65 reservoirs which had initial capacities in excess of 50,000 acre feet; this is about 45 percent of all the resurveys reported on large reservoirs by all agencies. The sedimentation data and the costs of the records are increasing at some geometric rate.

The writer believes that the results of the analysis and evaluation study, suggested above, would be an important advancement in ameliorating weaknesses in reservoir sediment investigation practices.

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SEDIMENT YIELDS
IN THE
UPPER MISSISSIPPI RIVER BASIN
by
Frank J. Mack[1]

INTRODUCTION

The U.S. Senate Committee on Public Works adopted a resolution on 21 May 1962 authorizing the Upper Mississippi River Comprehensive Basin Study. The North Central Division of the U.S. Army Corps of Engineers was assigned the responsibility of coordinating the UMRCBS. The Study was made with the full cooperative effort of Federal, State, and local agencies in their respective areas of interest. The final report is scheduled for submission to the Congress during 1971.

The basic objective of the UMRCBS was to prepare a general framework of development which will provide the best uses, or combination of uses, of water and related land resources. The main report is a coordinated summary of factual data, conclusions, and recommendations developed in 17 appendixes. Appendix G, Fluvial Sediment, provides a general appraisal of sedimentation and related problems in the Upper Mississippi River Basin. This appraisal was made because of the significance of sediment in the overall management of water and related land resources. This included the definition of the sediment problems; the areal distribution, magnitudes, variability and characteristics of sediment; and trends in sediment yields. The appraisal was based on available data from State and Federal agencies actively engaged in the collection of sedimentation data.

BASIN DESCRIPTION

The Upper Mississippi River Basin, shown in figure 1, drains an area of 18,000 square miles. The drainage system is comprised of portions of seven states in north-central United States. The basin is relatively low-lying and gently to moderately rolling in character. The present land surface was the principal result of glacial deposition. Following the retreat of the glacial ice sheets, windblown material was deposited on about half of the basin. This action formed the largest and most productive agricultural area in the United States.

Land Resource Areas, see figure 1, were used as the basis for grouping the sediment yield data for generalization of sediment yields presented in Appendix G. The dominant physical characteristics that define a Land Resource Area are: land use, elevation and topography, climate, water, and soil types. Sediment materials are derived primarily from soils and to a lesser extent from sand, gravel, and softer exposed bedrock. Areas with similar physical characteristics generally have similar rates of sediment production.

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About 75 percent of the Upper Mississippi River Basin is in the region commonly referred to as the "Corn Belt". This region is one of the outstanding grain producing areas of the world. It is also the highest sediment-producing region in the Upper Mississippi River Basin.

RECORDED SEDIMENT INVESTIGATIONS

Available data. The investigation of sediment in the Mississippi River began in 1838 when Captain A. Talcott made a number of sediment observations near the mouth of the Mississippi River. Since that time, numerous sediment investigations have been conducted by various persons interested in the sediment transport capacity of the Mississippi River. Notable among these were the sediment studies carried out by Humphreys and Abbott during the period 1850-1860 (U.S. Government, 1876). To further the objectives of collaboration and coordination, an Interdepartmental Committee on Sedimentation was established in 1939 by the National Research Council, Division of Geology and Geography. This committee initiated the first cooperative effort in standardization of equipment and practices in sedimentation investigations. In 1946 the Federal Inter-Agency River Basin Committee was organized and established a Subcommittee on Sedimentation to take over the functions of the original group. During 1955 the Subcommittee was placed under the direction of a technical committee of the Inter-Agency Committee on Water Resources. In 1966 these activities were placed under the auspices of the Federal Water Resources Council. The Subcommittee on Sedimentation early established a joint endeavor by several Government agencies to review the miscellaneity of equipment used in sampling the sediment transport capacity of streams and the dissimilarity of practies employed, both in field and laboratory analyses. The Subcommittee began to develop, by observation and laboratory tests, the relative value of results obtained in using such equipment and practices, and to develop standard methods for treating specific problems in sediment investigations.

The Corps of Engineers has made sedimentation surveys of the Mississippi River navigation pools since canalization of the Upper Mississippi River was completed in 1940. The measurement of sediment in reservoirs in Illinois and the evaluation of the various reservoir and watershed characteristics that affect sedimentation have been the subject of a comprehensive research project for the last 30 years by the Illinois State Water Survey. Reservoir sedimentation surveys have been made by various interests periodically in the Upper Mississippi River Basin during the past 40 years to measure quantitatively the soil losses from selected areas. The Federal program for such studies has been under the direction of the Soil Conservation Service since 1935.

Suspended sediment sampling in the Upper Mississippi River Basin has been conducted by the U.S. Geological Survey since about 1940. In many instances sediment investigations are made in cooperation with State agencies. Suspended sediment investigations were carried on by the Rock Island District, Corps of

Engineers, prior to 1940, but it was not until then that sampling stations were established on various streams within the District where it was apparent that the sediment load of the streams would affect the design, operation, or useful life of a proposed improvement. The initial sediment program has been expanded during the past 25 years as the need for additional suspended load data became apparent.

An inventory of available sedimentation data in the Upper Mississippi River Basin reveals that the collective coverage of the basin has been extensive. Sediment discharge records have been accumulated at 121 collection sites, and reservoir deposition has been determined by means of surveys at 132 locations. A study was made of this data, and, in order to establish a consistent means to compare the data, a 20-year base period between 1945 and 1964 was selected. All suspended sediment records and reservoir deposition surveys used have been related to the base period. Evaluation of sediment data for reservoirs included only impoundments created after 1930 unless the reservoir had been resurveyed. Some of the suspended load data were not used during the final evaluation of the records because of an insufficient sampling period. Data were also eliminated at some sites where sediment yields were affected by reservoir impoundments. Where the sediment load data were used at sites downstream from reservoirs, the yield was adjusted for the period of reservoir operation. A total of 83 suspended load stations and 57 reservoirs were ultimately selected for determination of sediment yields. Locations of these points are shown on figure 2. General sediment survey results for most of the reservoirs are published by the Inter-Agency Water Resources Committee (1964).

Computation of sediment yield from streams. Generally suspended sediment samples are taken to determine the sediment discharge of the stream and the particle-size distribution. It is essential to obtain at least some basic data on any stream where a sediment problem is to be studied. Computations based on meager data may be considerably in error, but they will generally be better than estimates based entirely on data from other streams, since sediment concentrations and also yields may vary widely from one stream to another.

Long-term suspended sediment records usually produce reliable estimates of sediment yields. Sediment concentrations, in parts per million, are plotted and connected by a continuous curve to obtain a concentration graph from which daily mean concentrations are determined. From the sediment concentration and the water discharge, the daily sediment load in tons is computed. From these data, monthly and yearly sediment yields are determined. For those stations that were operated during the 20-year base period of 1945 through 1964, the individual yearly suspended load values were averaged to determine the mean annual load.

When only a short-term sediment record was available on a stream with long-term flow records, an estimate of the long-term average sediment discharge was made by using the relationship of the available sediment discharge to water discharge and relating this to the long-term streamflow record (Jordan and others, 1964).

The method of computation consists of determining from the sediment discharge and water discharge relationship, the quantity of sediment discharge corresponding to a given increment of water discharge throughout the range of the flow-duration record for the stream. From the quantities of sediment discharge thus obtained, the average annual suspended sediment discharge was estimated.

No satisfactory method exists for routinely measuring the bedload. However, measurements and estimates by experienced observers indicate that the bedload ranged from 0 to 40 percent, generally being 10 percent of the total sediment discharge for streams in the Upper Mississippi River Basin. The bedload amount was added to the computed suspended sediment load to give the total sediment discharge. The adjustment of sediment discharge to the 20-year base period requires the following assumptions:

a. The relationship of the instantaneous sediment discharge to concurrent water discharge is similar to the relationship for daily values.

b. The sediment discharge to streamflow relationship existing during the period of field measurements represents the long-term relationship.

c. Water discharge records, as summarized in flow-duration computations are representative of the long-term flow regimen of the stream.

Changes in any of the important factors influencing sediment yield would alter the average relationship of sediment discharge to water discharge. Also, any extensive regulation of streamflow by the construction of storage reservoirs would affect the relationship.

Computation of sediment yield by reservoir deposition surveys. The total sediment yield of a drainage area during a long period of time can be computed from the volume of sediment deposited in a reservoir. The volume of accumlated sediment is measured by a survey. Specific weight of the sediment in place is determined to compute the weight of the material deposited. Surveys of sediment deposits are made by taking cross sections of the reservoir at spacings sufficiently close to permit computation of the volume of the deposit. An adjustment was made for trap efficiency. The volume of sediment that is deposited or trapped is dependent on the trap efficiency of the reservoir. Sediment accumulation in a reservoir was also adjusted to the base period by a direct ratio of the period between surveys. From these determinations the average annual sediment yield was computed.

Detailed explanations for making a reservoir sedimentation survey are given by Eakin and Brown (1939) and by Gottschalk (1949). Reservoir surveys should: (1) measure as accurately as possible the total volume of sediment accumulated, its distribution, its specific weight, its relation to capacity and age of the reservoir, and to size of drainage area; and (2) establish a permanent system of monuments which future surveys may utilize for comparing variations in sedimentation rates as reflected in a change of erosional conditions, either from acceleration or retardation through erosion control or the lack thereof.

SEDIMENT YIELDS

Relative magnitudes. The principal results of this report are shown graphically in figure 3. These curves represent the specific findings of this study and represent the best information available as to the sediment yields expected throughout the Upper Mississippi River Basin. The curves are supported by the comprehensive and detailed analysis of all the sedimentation data available for the basin, as analyzed by the task force preparing this report.

The Land Resource Areas (see figure 1) were used as the basis for the analysis of relative sediment yields in the basin. Each solid line in figure 3 represents the sediment yield expected for drainage areas of various sizes within a particular Land Resource Area and shows the changing relation between sediment yield and drainage area size. The position of the various lines in figure 3 is an indication of the relative magnitude of the sediment yields of the various Land Resource Areas.

From figure 3 it can be seen that sediment yield rates vary considerably throughout the basin. Generally the sediment yield is approximately 200 to 250 times greater in the extreme southern portion of the basin in comparison with the yields in the extreme northern part of the basin. The highest solid line in figure 3 is that representing Land Resource Area 115 in the southern (downstream) portion of the basin. The lower most curve in the figure is for Land Resource Areas 88, 90 and 93 in the extreme northern (upper) part of the basin.

The two dashed lines represent special conditions along bluffs in Land Resource Areas 103 and 115. These dashed lines represent much higher sediment yields for the bluffs along the Des Moines River above Des Moines, Iowa, and the Mississippi River in Missouri and Illinois, than yields from the flatter uplands of these Land Resource Areas.

The curves in figure 3 slope downward to the right because the amount of sediment discharge from a large drainage area is less on a per-square-mile basis than the sediment discharged from a small drainage area. This decrease in sediment yield as the drainage area increases was found to be present in all of the data analyzed.

As the size of the drainage area increases, the rate of sediment production per square mile decreases. This is the result of several factors. The chances that an intense storm will occur over the entire watershed become less and less as the watershed becomes larger. There is more variation in the rate of sediment production in smaller watersheds due to the fact that all physical factors vary more widely. The percentage of area of steep surface slopes with correspondingly high erosion rates decreases with the larger drainage area, which would tend to decrease the sediment yield per square mile of the larger basins. (Leopold and Maddox, 1953, p. 22.)

However, for some streams the sediment load per square mile of drainage area actually increases as the size of drainage area increases due to the nonuniform physiography and land use of a basin. The main stem of the Mississippi River is an example. From Wabasha, Minnesota to Hannibal, Missouri, the estimated long-term annual rate of sediment production increases from 4 to 181 tons per square mile. This increase is due primarily to changes in land use, annual runoff, soil types, and topography. For the drainage area as a whole, the proportion of land in cultivation increases by 30 to 40 percent between these two points and the runoff per square mile is also markedly increased. Above Wabasha there are a great many lakes that intercept the sediment. Below that point there is a marked change in soils and topography, as drainage from the driftless area enters the Mississippi River.

To develop figure 3, the adjusted sediment yields, in tons per square mile per year, for each station and each reservoir surveyed were plotted against drainage area, by Land Resource Areas. Figure 4 shows the plot for Land Resource Area 108. This area serves as a good example since it is among those having data available from all agencies participating in this study. The data cover a wide range in sizes of drainage areas, and include both reservoir and stream sediment yields.

A curve was fitted to the plotted data as shown on figure 4. The individual points, particularly those which plotted outside of the general grouping of points, were evaluated separately. For example, the location of points S-68, 69, and 70 on figure 4 indicate that these streams have sediment yields higher than most streams in Land Resource Area 108. These streams, the Middle River, South River, and Whitebreast Creek, all in south-central Iowa, lie along the western part of Land Resource Area 108 and adjacent to an area of higher sediment yields. Therefore, these streams are not considered to have sediment yields typical of Land Resource Area 108 and were given less weight in locating this average yield curve. The curve is defined by the exponential relationship

$$Y = K/A^{n}$$

where Y is the sediment yield, K is the value of the sediment yield from one square mile of drainage area, A is the drainage area in square miles, and n is the exponent establishing the slope of the curve. A value of 0.12 for n was found to give the best fit. This value for the slope of the curve further provided the best fit for a natural grouping of curves for all 17 of the Land Resource Areas in the Upper Mississippi River Basin into the 9 sediment yield curves shown as solid lines on figure 3.

Table 1 shows the relationship of the average annual sediment yield by Land Resource Areas, for drainage areas of 1, 10, 100, 1,000, and 10,000 square miles. These data are taken from the 11 sediment yield curves in figure 3 and represent the primary results of this study.

Sediment yields in the upper part of the Mississippi River Basin are relatively low and increase toward the lower or downstream end of the basin, as shown by the map in figure 5. The sediment yield from a drainage area of

100 square miles in the Upper Mississippi River Basin is shown. The sediment isograms have been drawn on a generalized basis according to the sediment yield for the individual Land Resource Areas and represent only the sediment yield for a 100-square mile drainage area. Sediment isograms for other sizes of drainage areas would have the same general pattern. The data presented in this form are only for illustrative and informational purposes and should not be used to determine specific sediment yields.

TABLE 1

SEDIMENTATION RATES

ANNUAL YIELD IN TONS PER SQUARE MILE

Land Resource Area	Drainage Area in Square Miles: 1	10	100	1,000	10,000
88, 90, and 93	15	12	9	7	5
91	45	34	26	20	15
102 and 103	76	58	44	33	20
95, 98, and 110	91	70	52	40	30
104 and 116	450	340	260	200	150
103 (Bluff drainage)	560	410	--	--	--
105	910	690	520	400	300
113	1,200	910	690	520	400
108, 109, and 114	1,500	1,100	860	660	500
115	3,600	2,800	2,100	1,600	1,200
115 (Bluff drainage)	11,000	8,000	6,000	--	--

By using the curves in figure 3, it is possible to derive figure 6, which shows for a given drainage area, the relative sediment yields throughout the basin. In order to construct figure 6, the sediment yield for a drainage area of 35 square miles was read from figure 3 for each of the Land Resource Areas.

The Land Resource Areas which occur in the upper end of the basin are shown at the left on figure 6 and those which occur at the lower end of the basin are shown at the right. This figure shows again that the sediment yields increase as we go from the upper (northern) end of the basin to the lower (southern) end of the basin.

The great range in the magnitude of sediment yield from the various Land Resource Areas in the Upper Mississippi River Basin is also apparent. For a drainage area of 35 square miles, the lowest sediment yield, 10 tons per square mile per year, occurs in Land Resource Areas 88, 90, and 93. For Land Resource Areas 95, 110, and 98, the sediment yield amounts to 60 tons per square mile per year. The adjoining sediment yield value represents that for Land Resource Areas 104 and 116, at 300 tons per square mile per year. The expected sediment yield jumps to 1,000 for Land Resource Areas 108, 109, and 114. The highest sediment yield shown in figure 6 is 7,000 tons per square mile, which is expected from the bluff areas in Land Resource Area 115.

Trends in sediment yield. Sediment yields of streams vary greatly from year to year due to the large variation in the number, intensity, and type of rain storms that occur in a basin each year. This large natural variability in annual sediment discharge makes the detection of meaningful time trends difficult and requires detailed analysis of long-term records of sediment discharge. In the Upper Mississippi River Basin, sediment data are not sufficient to define a general trend, if any, in the sediment yield from the basin. However, the few long-term records that are available were reviewed and tested for trends in yield.

With the emphasis and widespread application of soil conservation practices, it is logical that a measurable reduction in the sediment yields from the watersheds would result. Statistical analyses of the long-term sediment records of the Upper Mississippi River Basin showed that a decrease in sediment yield was indicated for some areas, while in others, no trend was evident.

MISSISSIPPI RIVER MAIN STEM

The Upper Mississippi River is generally considered to be a clearwater stream and may be so classed until it is joined by the Missouri River. At this junction the Upper Mississippi receives large quantities of sediment from the Missouri River, making it, below this point, necessary to regard the main stem as a heavy sediment carrier. However, this broad classification of the Upper Mississippi River as a clearwater stream is only relative and the fact remains that this portion of the river does transport annually over 25 million tons of sedimentary material. The quantity of material transported by the Upper Mississippi River is of large enough magnitude to be an important factor in maintaining the present 9-foot depth channel navigation development by means of dredging operations and open river regulating works.

The average daily sediment load transported by the Upper Mississippi River is about 500 tons per day in the vicinity of St. Paul, Minnesota. The average daily sediment load of the Mississippi River at Hannibal, Missouri, 115 miles above the mouth of the Missouri River, is about 70,000 tons per day. At St. Louis, Missouri, below the Missouri River junction, the load averaged about 500,000 tons per day during the period 1948-1958 (Jordan, 1965). This indicates that approximately 14 percent or 25 million tons of the 181 million tons of sediment transported annually by the Mississippi River at St. Louis, Missouri, is attributable to the Upper Mississippi River Basin.

CONCLUSIONS

Sediment problems caused by excessive yields in the Upper Mississippi River Basin are major and deserve attention by the public, by professionals, by administrators, and by legislative bodies.

In order for sediment information to be useful in the comprehensive planning of water resources development for the Upper Mississippi River Basin, it is necessary that the sediment yields be interpreted in some manner to show what sediment problems are created by these yields. Figure 3 illustrates the wide range in sediment yield throughout the Upper Mississippi River Basin. The low magnitude of yields in the northern part of the basin indicates that few sediment problems would be created, although there are undoubtedly certain exceptions to this general conclusion. It is easily apparent that the high yields in the southern part of the basin can create serious sediment problems and that this region is greatly contributing to the sediment problems in the Upper Mississippi River Basin.

In between the range in sediment yields, the lower of which seemingly creates few problems, and the other creating serious problems, there must be a level of sediment yield which could generally be labelled as tolerable. The establishment of a tolerable level of sediment yield is difficult; it involves considerable subjective judgment; and it is subject to many exceptions. However, in order to clarify the size and extent of sediment problems in the Upper Mississippi River Basin, and in order that sediment data have maximum usefulness in comprehensive planning, a tolerable level of sediment yield has been established. Based on a reasonable judgment, the sediment yields in the larger drainage areas of the Upper Mississippi River Basin might be considered tolerable if they do not exceed 500 tons per square mile annually from a 100-square mile drainage area. Sediment yields higher than this would be considered excessive, and consequently should be reduced. It should, however, be recognized that the subjective assumption of a tolerable level of sediment yield of 500 tons per square mile annually from a 100- square mile drainage area is only useful as a means of quantifying the broader sediment problems of the basin.

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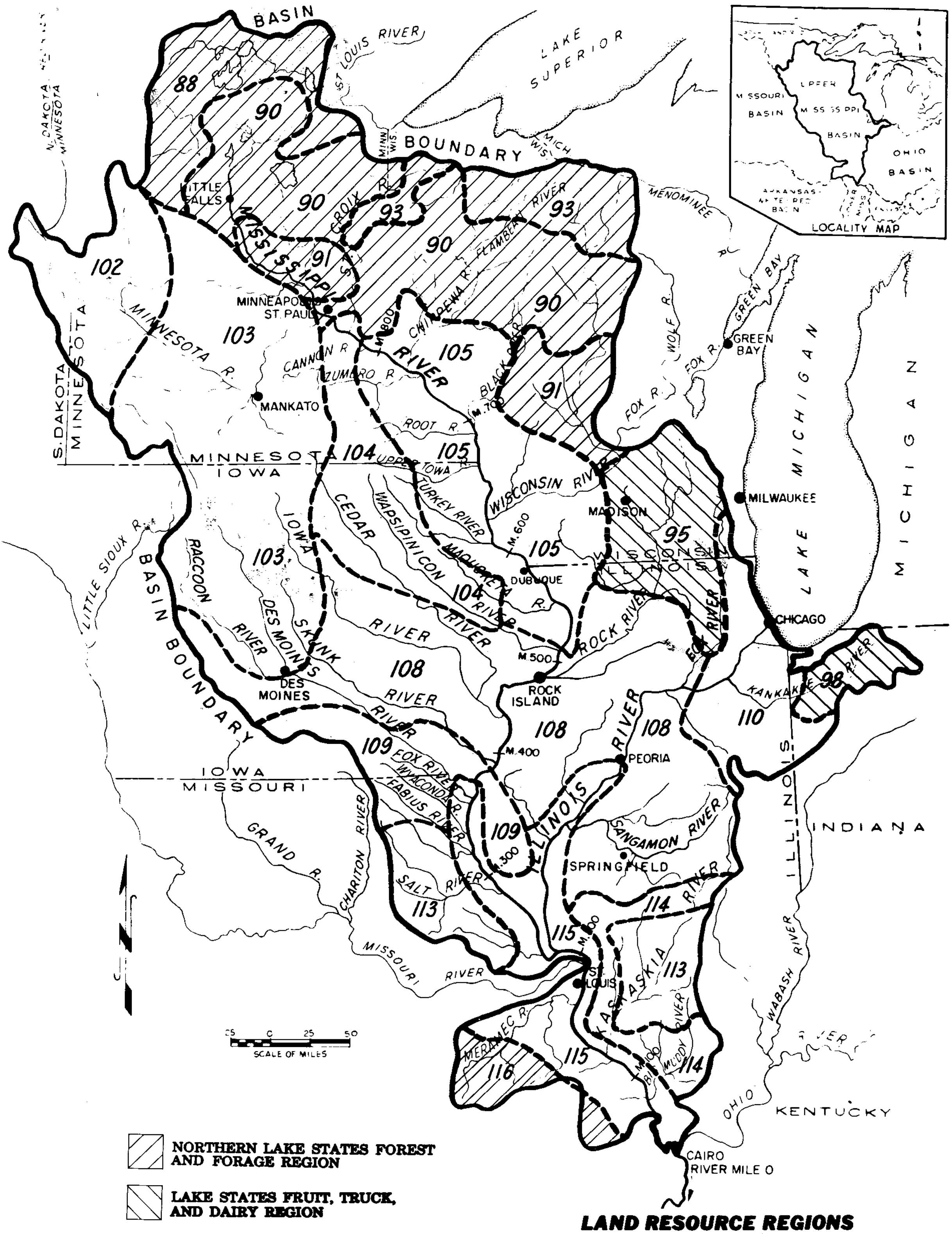


NORTHERN LAKE STATES FOREST AND FORAGE REGION

LAKE STATES FRUIT, TRUCK, AND DAIRY REGION

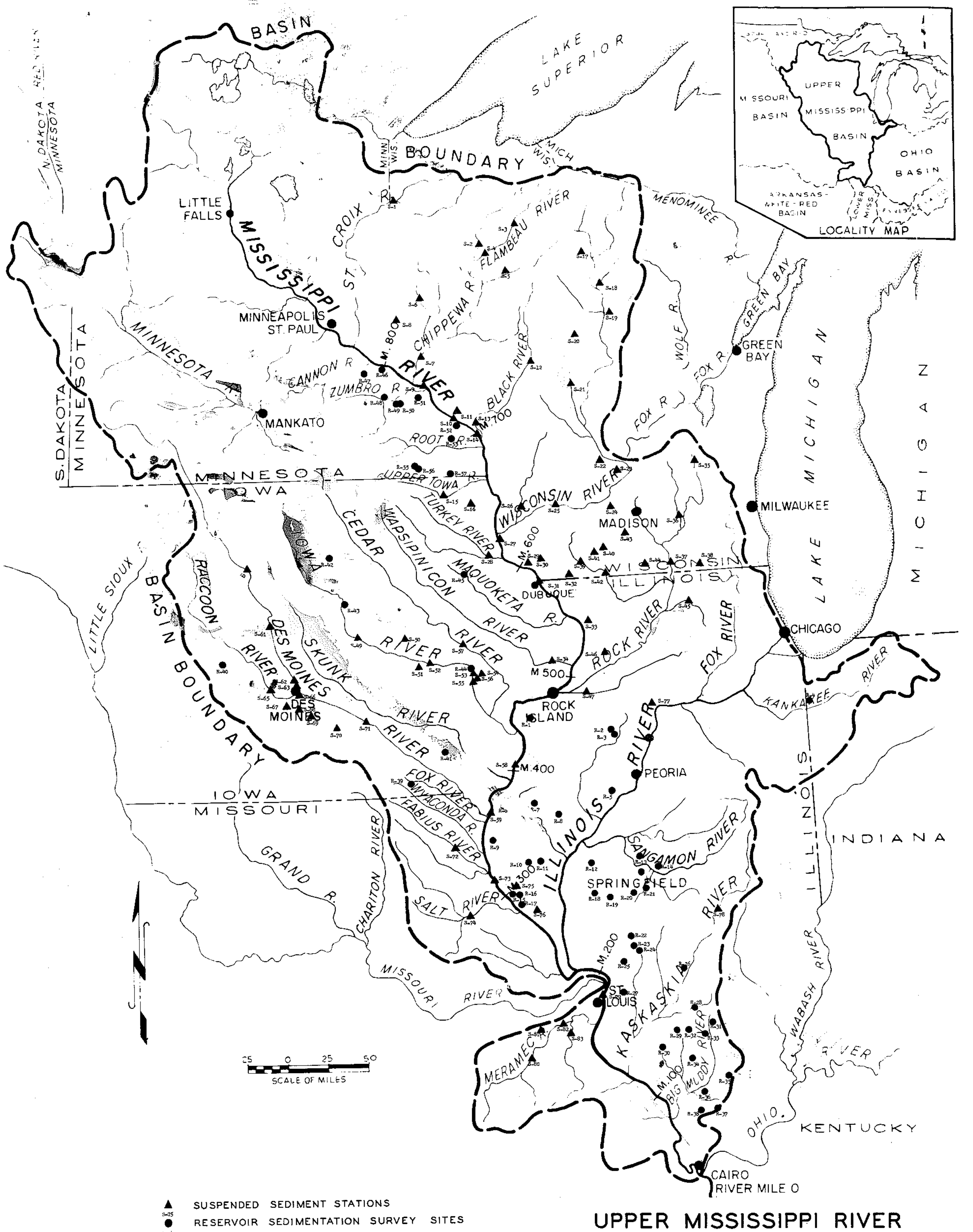
CENTRAL FEED GRAINS AND LIVESTOCK REGION

EAST AND CENTRAL GENERAL FARMING AND FOREST REGION

LAND RESOURCE REGIONS AND MAJOR LAND RESOURCE AREAS

UPPER MISSISSIPPI RIVER
COMPREHENSIVE BASIN STUDY
APPENDIX G, FLUVIAL SEDIMENT

Figure 1



▲ SUSPENDED SEDIMENT STATIONS

● RESERVOIR SEDIMENTATION SURVEY SITES

UPPER MISSISSIPPI RIVER
COMPREHENSIVE BASIN STUDY
APPENDIX G, FLUVIAL SEDIMENT

LOCATION OF BASIC DATA

Figure 2

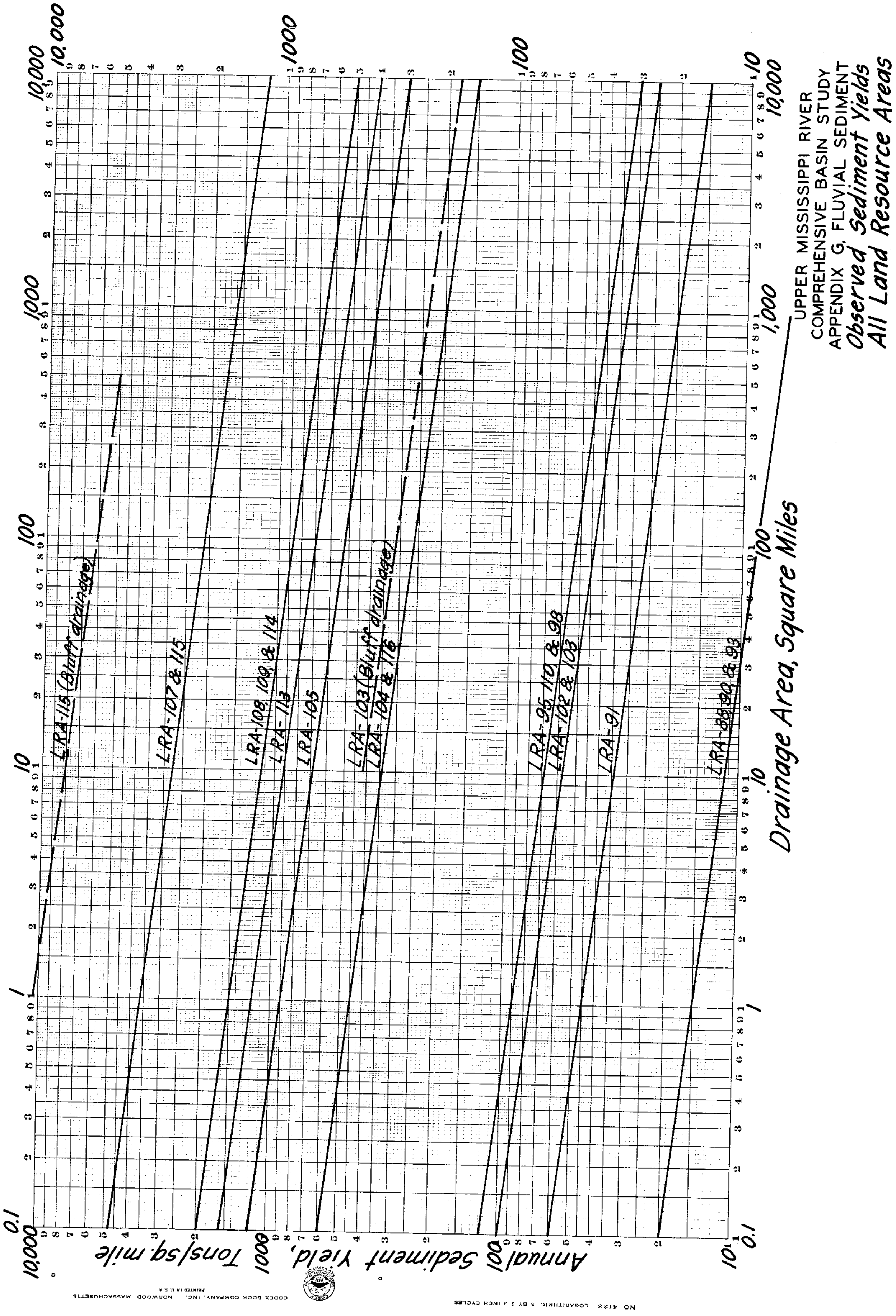

UPPER MISSISSIPPI RIVER
COMPREHENSIVE BASIN STUDY
APPENDIX G, FLUVIAL SEDIMENT
Observed Sediment Yields
All Land Resource Areas
Figure 3
Drainage Area, Square Miles
Annual Sediment Yield, Tons/sq. mile
LRA-115 (Bluff drainage)
LRA-107 & 115
LRA-108, 109, & 114
LRA-113
LRA-105
LRA-103 (Bluff drainage)
LRA-104 & 116
LRA-95, 110, & 98
LRA-102 & 103
LRA-91
LRA-88, 90, & 93
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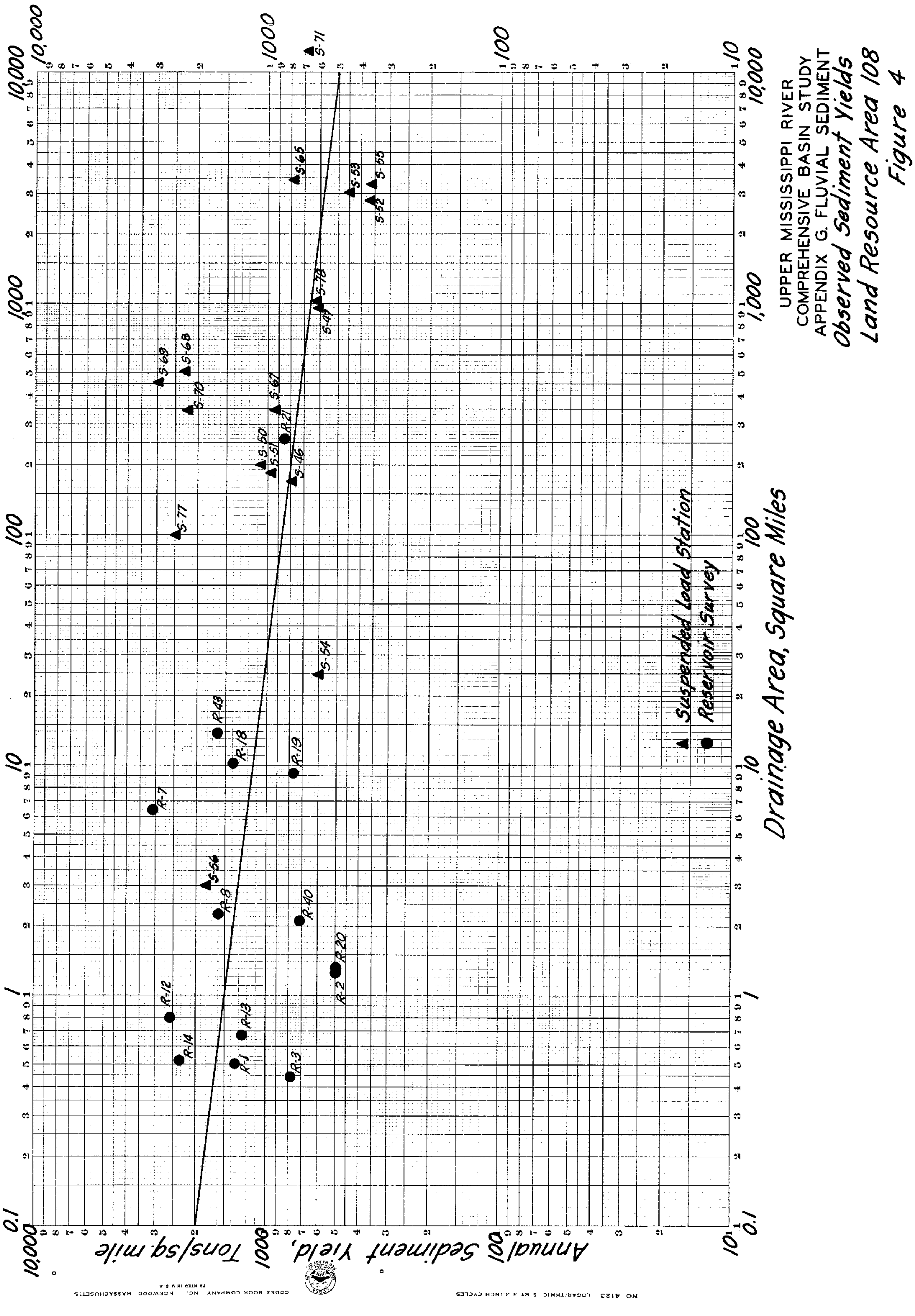
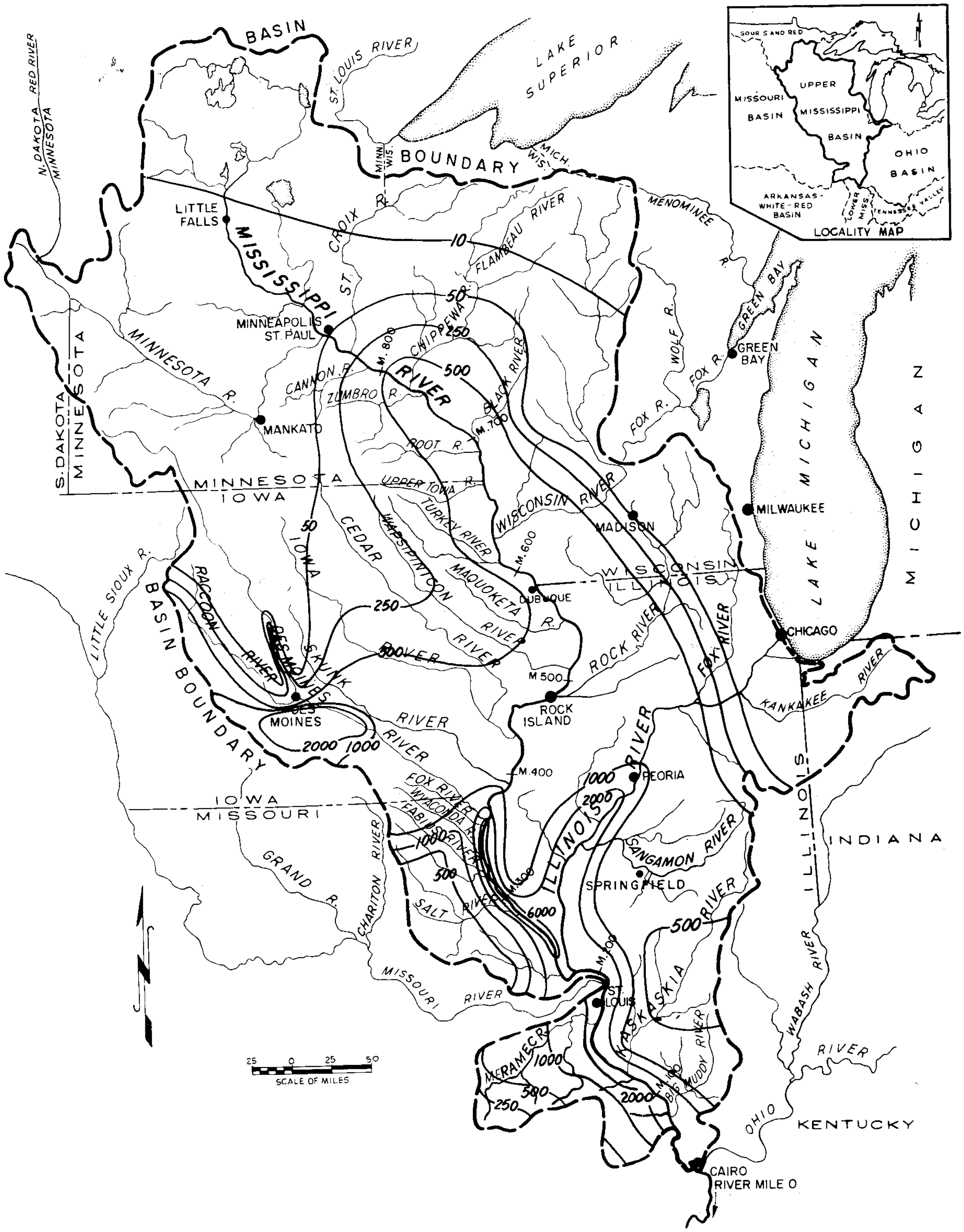
Figure 3



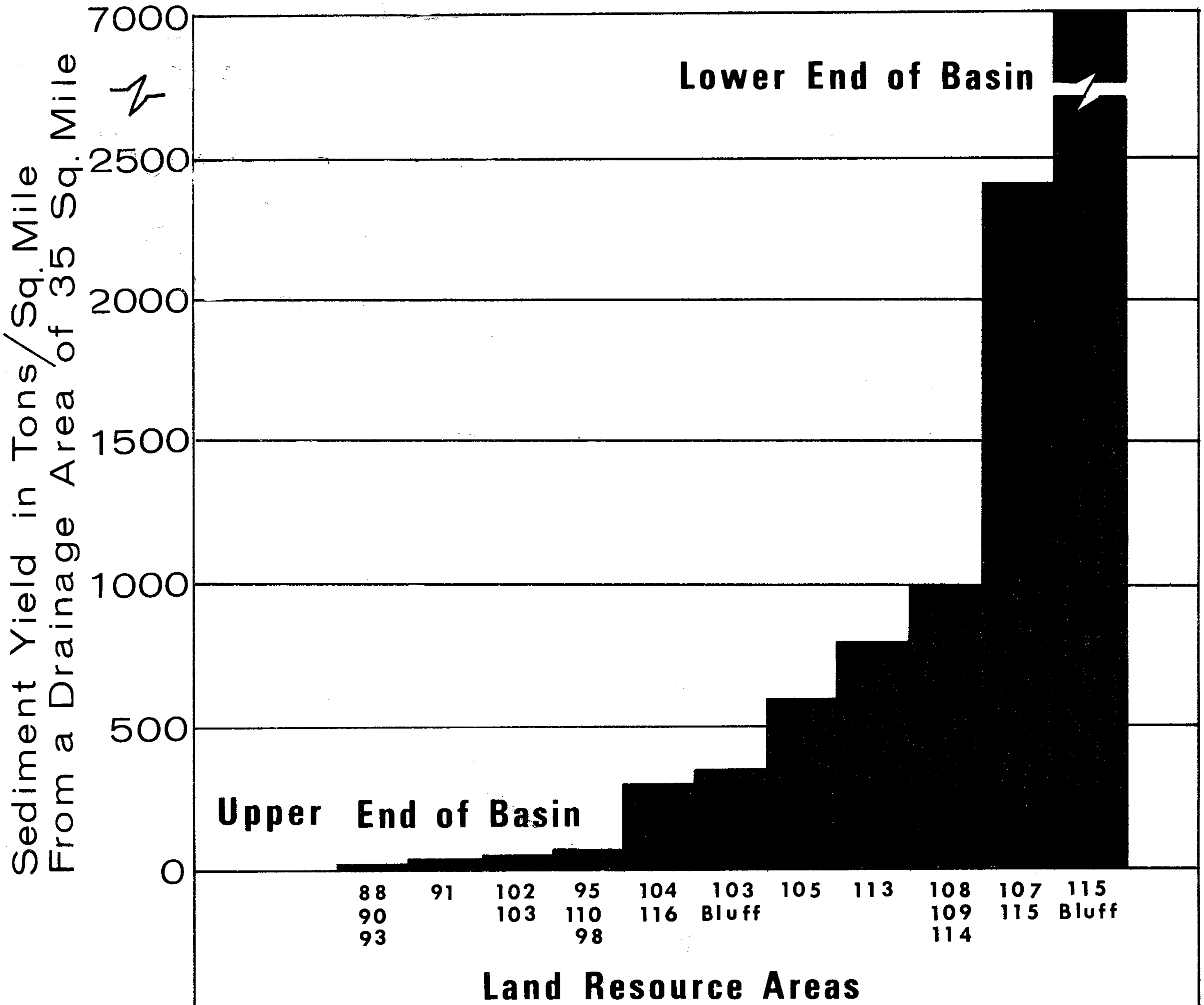
UPPER MISSISSIPPI RIVER
COMPREHENSIVE BASIN STUDY
APPENDIX G. FLUVIAL SEDIMENT
Observed Sediment Yields
Land Resource Area 108
Figure 4
Drainage Area, Square Miles
Annual Sediment Yield, Tons/sq. mile
Suspended Load Station
Reservoir Survey
S-71
S-65
S-53
S-55
S-52
S-78
S-49
S-69
S-68
S-70
S-67
R-21
S-50
S-51
S-46
S-77
S-54
R-43
R-18
R-19
R-7
S-56
R-8
R-40
R-20
R-2
R-12
R-13
R-14
R-1
R-3

Figure 4



UPPER MISSISSIPPI RIVER
COMPREHENSIVE BASIN STUDY
APPENDIX G, FLUVIAL SEDIMENT

Annual Sediment Yield
For 100 sq. mi. Drainage Area
in Tons Per Sq. Mile
Figure 5



Relative Sediment Yields In The Upper Mississippi River Basin. Representative Yields Shown Are Those Expected From a 35 Square Mile Drainage Area In The Various Land Resource Areas.

Figure 6

SEDIMENTATION STUDY
PROPOSED MULTI-PURPOSE RESERVOIR

by

Elmer W. Gable[1]

1. Scope. This study contains preliminary investigations made on sedimentation in the proposed reservoir to determine whether or not a substantial delta will form. Data in Corps of Engineers reports and studies by the State on sedimentation for Lake Decatur were utilized.

2. General Nature of the Study. The natural flows and sediment loads were estimated. Suspended loads were derived from data for Lake Decatur. Bed load is negligible. Sediment concentrations in the backwater reaches were estimated from the average velocities and depths in the various reaches. From the concentrations of sand, silt and clay the depths of deposits in the reaches were estimated. The distribution of sediments in the proposed reservoir for a 100-year period was estimated by two alternate methods.

3. Sediment load. The Lake Decatur sediments consist mainly of poorly compacted silt and clay. The sediment is fine in texture and size is carried into the reservoir as wash load. A small amount of sand is present, probably from bank washings. Sedimentation studies by the State of Illinois for Lake Decatur show that the average annual volumes deposited in the lake were:

1922 - 1936	198 acre-feet[a]
1936 - 1945	236 " "
1946 - 1956	77 " "
1956 - 1966	103 " "

[a]See reference 1.

The average for the period 1922 to 1966 was 165 acre-feet annually. (See plate 1). The low value for 1946-1956 is due somewhat to drying of sediments during the droughts in this period. The rates since 1946 are lower than those prior to 1946 and may be due to a natural decrease of sediment production from the watershed and the result of efforts directed toward better soil conservation. It was assumed that 100 acre-feet annually would accumulate for a 100-year period.

4. For the 10-year period 1936-1945 the total tonnage reported was 2,650,000 tons[2]. The average weight of sediment is $\frac{265,000}{236 \times 43,560} = 0.026$ tons/cu.ft.

or 52 lbs./cu.ft. A percentage size distribution and unit weights corresponding to the Keokuk, Iowa pool was assumed[3]:

Portion	Percent by weight	Weight per cu.ft.
Sand	10	93
Silt	53	65
Clay	37	30

---

[1] Hydraulic Engineer, Chicago District

The adjusted unit weights for the proposed reservoir are as follows:

Portion	Weight in 1 cu. ft. lbs.	Volume cu. ft.	Adjusted to 1 cu. ft.	Adjusted weight lbs./cu. ft.
Sand	5.2	.056	.05	104
Silt	27.6	.425	.38	73
Clay	19.2	.640	.57	34
	52.0	1.121	1.00	

The estimated average annual deposit in tons is estimated at 113,260 tons, or 310 tons per day.

Portion	Tons Year	Tons ac. ft.	Volume ac. ft.
Sand	11,330	2,265	5.0
Silt	60,030	1,500	37.8
Clay	41,910	740.5	56.6

The average annual suspended sediment load consisting of 5 ac. ft. sand, 38 ac. ft. silt and 57 ac. ft. clay was used for calculations.

5. The trap efficiency was estimated from the ratio of reservoir capacity to average annual inflow. The average annual discharge at a point upstream of the proposed reservoir for a 58-year period was 386 c.f.s. At the dam the average flow is estimated at 565 c.f.s. or 409,000 ac.ft. annually. The proposed reservoir capacity is 90,600 ac. ft. The trap efficiency corresponding to the ration, 0.22 is about 94 percent for medium sediments[5]. Estimates by the State of Illinois show that the trap efficiency of Lake Decatur is 78 percent[2]. The tentative operation plan requires a uniform release of 5,000 c.f.s. during floods which will carry sediments from the reservoir. The trap efficiency would be somewhat higher than the lake Decatur value. All flood flows pass over the dam at Decatur.

6. Flow duration. From natural flows of the stream, drainage area 550 sq. mi., flows at proposed dam, drainage area 750 sq. mi., were estimated. The durations at the dam are:

Percent of time equalled or exceeded	Reservoir inflow c.f.s.
40.7	280
30.0	480
20.0	750
8.8	1,500
.4	6,500
.1	13,000

A series of sediment samples was taken a short distance upstream of the reservoir to determine open river concentration during the early stages of the project. However, due to priority of military work, the samples were not analyzed and were destroyed.

7. Sediment-discharge rating curve. Adequate data relating the suspended sediment load to various discharges under natural conditions is not available. One such measurement was made on May 4, 1936 after a 1.85" rainfall on May 2. Water samples were taken by State personnel at the head and outlet of the lake. The maximum suspended solids, measuring 530 p.p.m., was near the head of the lake and the minimum of 50 p.p.m. was near the outlet. The flow, estimated at 3,240 c.f.s., indicates a sediment load of 4,600 tons per day for this flow. The State estimates that most of the sediments are carried by one percent flows and less. Therefore, only the three highest flows in paragraph 6 were assumed to carry sediment. The weighted sediment loads were assumed to be proportional to the weighted flows as follows:

Percent of time	Q c.f.s.	Weighted c.f.s.	Weighing c.f.s.	Weighted load tons/day	Open river load tons/day
8.8	1,500	132	.772	239	2,715
.4	6,500	26	.152	47	11,750
.1	13,000	13	.076	24	24,000
		171	1.000	310	

The above data is plotted on plate 2.

8. Analytical step method. When sediment enters the slack water of the reservoir the sand fraction is deposited first followed by the silt and clay sizes. The rate and location of the deposits depends on the discharge at the time, the sediment sizes, and the changes in velocity and depths of water across the floway. The proposed reservoir was divided into reaches for which the depths, velocities, slopes and other hydraulic elements were determined for various discharges by backwater computation. The sediment load was then related to the velocity and average depth. The amounts and locations of the deposits were computed by a sediment inflow-outflow procedure from reach to reach. At the end of a selected period, say 10 years, the backwater computations should be repeated to obtain new hydraulic elements for the proposed reservoir reaches based on the accumulated deposits at the time. This method involves a large amount of detailed work and various assumptions which affect the reliability of the estimates.

9. Backwater profiles. Backwater computations for 1,500, 6,000, and 13,000 c.f.s. were made using HEC program 22-J2-L212, 1967 with a Control Data 6600 computer. Pool elevations were determined by flood routing through the reservoir. The joint use pool elevation 623.0 feet was used for 1,500 c.f.s. which is a 2-year flow. Pool elevation 636.0 feet was used for 13,000 c.f.s. which is a

10-year flow. Backwater profiles were also computed for the above flows under natural conditions to determine the extent of backwater effects.

10. Sediment load related to hydraulic elements. The concentration of the suspended load and hydraulic elements were obtained for several rises in backwater reaches of Garrison, Fort Peck and Fort Randall Reservoirs. The results of this study were used in design memorandum for sedimentation for Dardanelle Reservoir.[3] Plate 3 is a reproduction of plate 24 in that memorandum and was used in this study. The plate relates the relative concentrations of sand, silt and clay at a point in the backwater reach to the ratio of the Froude number at the most upstream point in the backwater reach. From backwater computer outputs the total flood areas, mean velocities, top widths and depths were calculated for the reach midpoints. The ratios of the mean velocities to the square root of the mean depths were computed for each midpoint. The ratios at or near the beginning of the backwater effects were selected and then the ratios of the Froude numbers for each reach. The corresponding $\frac{C_N}{C_1}$ ratios were read from plate 3. $\frac{C_N}{C_1}$ ratios less than 1.0 indicate a change from natural conditions and depositing of sediment in the reaches.

11. Deposits in channel and flood plain. To simplify calculations it was assumed that the sediment will deposit uniformly over the entire flooded areas. However, the channel is expected to have deeper sediment deposits than the inundated flood plain. This phenomenom has also been observed in Lake Decatur. Calculations for deposits in channel and flood plain were made for sand, silt and clay. For example, the sand portion is 5 percent of 100 ac. ft. per year, or 5 ac. ft. per year. The weighted C_1 values are proportional to the weighted flows according to paragraph 7.

Percent of time	Q c.f.s.	Weighted Q	Weighing factor	Weighed C_1 ac.ft./yr.	C_1 ac. ft./ yr.
8.8	1,500	132	.772	3.86	44
.4	6,500	26	.152	.76	190
.1	13,000	13	.076	.38	
		171	1.000	5.00	

The backwater effects begin near mile 164. Sample calculations for sand deposits in the first 3 reaches follow:

Mile	Q c.f.s.	$\frac{C_N}{C_1}$	C_1 weighed by % of time ac. ft. per yr.	C_N ac. ft. per yr.	$C_1 - C_N$ ac. ft. per yr.	Deposits: Upstream reaches ac. ft. per yr.	Deposits: In reach ac. ft. per yr.
	1,500	1.00	3.8	3.8	0	0	0
164.32	6,500	.16	.8	.1	.7	0	.7
	13,000	.45	.4	.2	.2	0	.2
							.9
	1,500	1.00	3.8	3.8	0	0	0
163.72	6,500	.025	.8	0	.8	.7	.1
	13,000	.073	.4	0	.4	.2	.2
							.3
	1,500	1.00	3.8	3.8	0	0	0
163.12	6,500	.006	.8	0	.8	.8	0
	13,000	.022	.4	0	.4	.4	0
							0

$C_1 - C_N$ is the total amount of deposit in the reaches upstream from section N in the backwater reach. Calculations proceed upstream reach by reach to find the deposition or erosion in each reach.

Similar calculations were made for silt and clay.

The consolidated deposits and average depths for one year and 10 years after closure were obtained.

Areas were weighted according to the 3 flows shown. Conditions 10 years after closure are shown graphically on plate 4. Between miles 159 and 160 scour averaging 1.5 feet is indicated. As shown by backwater computer output this is due to an increase in average velocity in this reach.

12. Sediment distribution. The distribution of sediment in the proposed reservoir after a 100-year period was estimated by the area increment and the empirical area reduction methods. The results are shown on plate 7. The area increment method shows that 65 percent of the 100-year sediments will deposit above the joint use pool. However, the empirical area reduction method shows that only 45 percent will deposit above the joint use pool. The distribution is similar to that in Lake Deeatur for the period 1923-1966[6]. These distributions along the lake axis and by elevation are shown on plates 5 and 6. In the first 34 years of existence the lake had lost 30 percent of its original capacity. An outstanding characteristic of sedimentation in the lake is the relative uniformity in thickness and types of deposits over the lake basin. The thickest deposits are in the main channel.

13. Discussion. Plate 4 shows that 70 percent of the 10-year sediments will deposit in the upper half of the proposed reservoir, For an 8-mile reach from the dam the deposits would be negligible. However, this study does not account for settling velocities of the finer material. Some of this material would be carried further downstream and would deposit in this reach. The deposits begin at mile 164 where the stream bed elevation is 623 feet, the joint pool elevation. The deposits will cause the velocities through the reduced flowways to rise which will move the sediment further into the proposed reservoir. The results of this study are preliminary since the step method should be repeated several times to show the sedimentation pattern. However, it is believed that a substantial delta will not form due to the low sand content. The sand deposits at the head of the reservoir would not be moved by future flows. A heavy sand content would cause a delta to form rapidly. No typical delta deposits have formed at the head of Lake Decatur, but small deltas in the two major tributaries Sand Creed and Big Creek have filled the original stream channels for some distance. However, the posibility of a formation of mud flats and swamps to some relatively small degree by shoaling at the head of the proposed reservoir after a long period of time should be recognized. Such deposits exist at the head of Lake Decatur. A series of 20 colored slides are available which show these formations. The sediment load leaving the proposed reservoir will be reduced. However, this is not expected to cause degradation downstream because the river does not carry a large bed load and this is a measure of its capacity to erode existing bed material. Furthermore, the higher discharges which now move most of the sediment will be controlled. The tentative operation plan requires a uniform release of 5,000 c.f.s. The effect of this on trap efficiency has not been resolved. Alternate project plans which reduce impact on the local ecology are being studied.

14. Conclusions. The sediment has a low percentage of sand. The preliminary results of the study show that a substantial delta will not form. The step method should be repeated several times to establish the pattern of deposits. The sediment is expected to deposit in a blanket of fine material over the entire bottom similar to the pattern in Lake Decatur.

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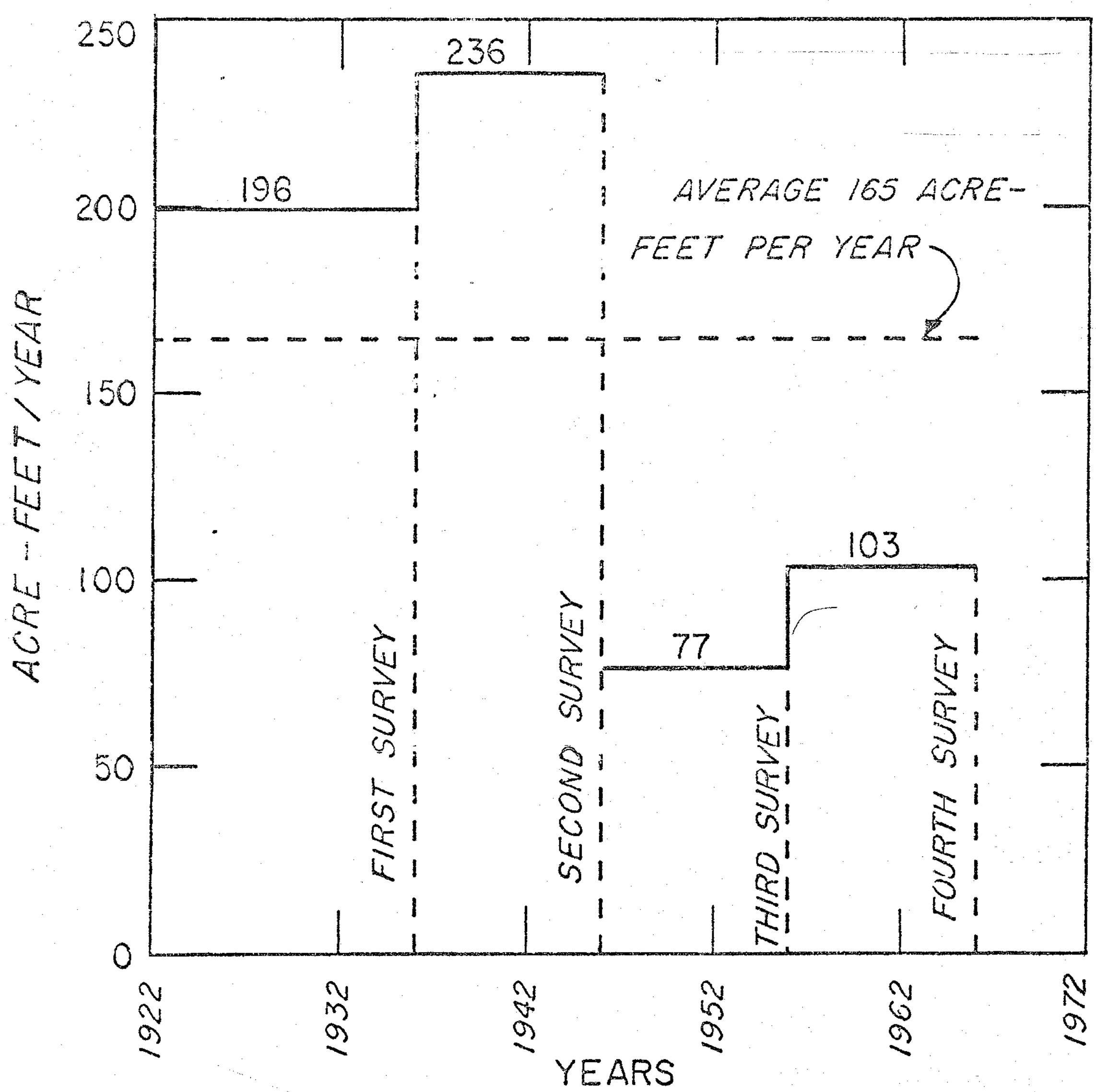
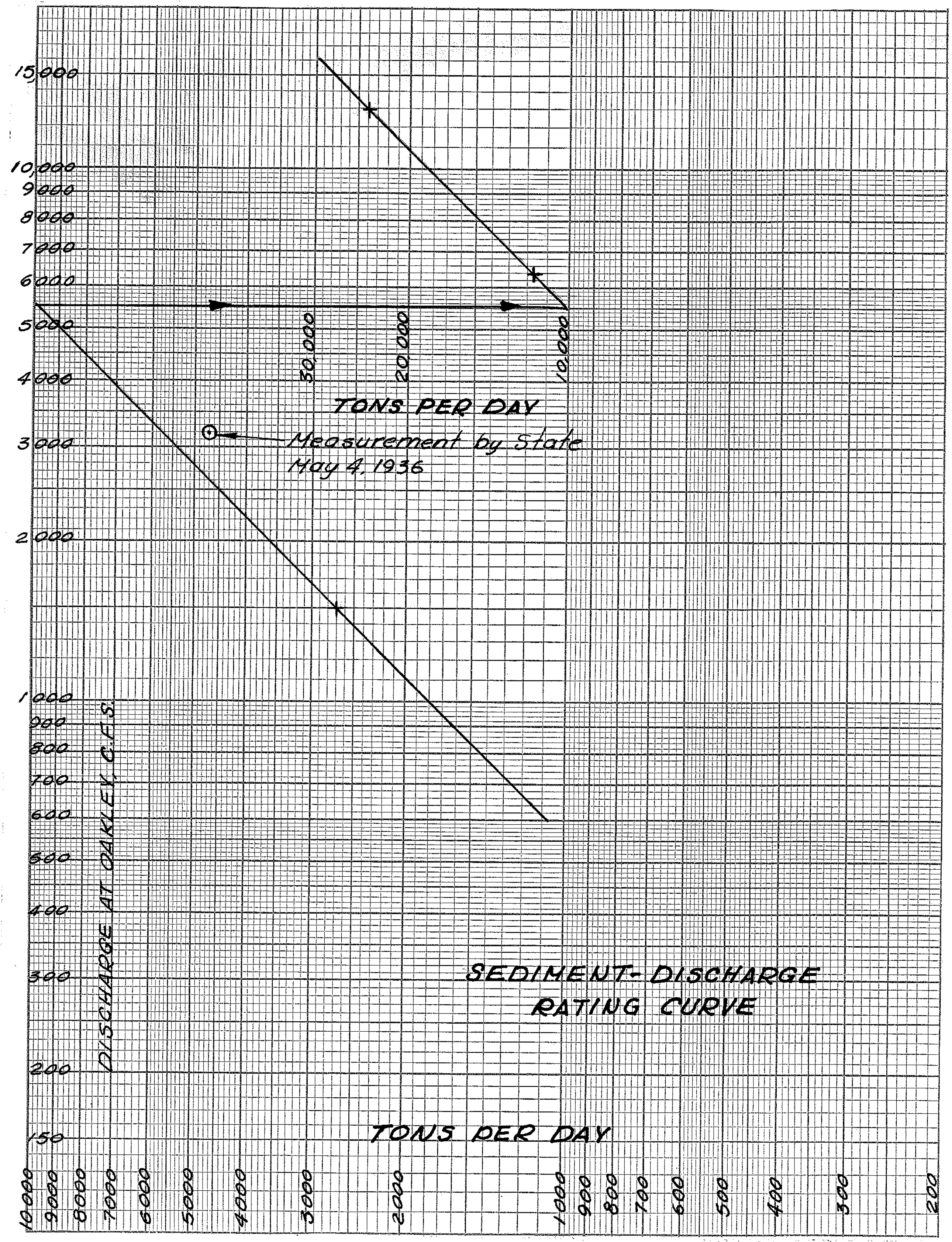
3.	Corps of Engineers, Little Rock District. "Project Design Memorandum No. 6, Sedimentation - Part IV Dardanelle Reservoir," - October 1957
4.	Lloyd C. Fowler, "Determination of Location and Rate of Growth of Delta Formulations," - Corps of Engineers, Missouri River Division
5.	Brune, G. M. "Trap Efficiency of Reservoirs," - Transactions American Geophysical Union, Vol. 34, No. 3 - June 1953
6.	Stall, John B. "Sediment Movement and Deposition Patterns in Illinois Impounding Reservoirs," - June 1964

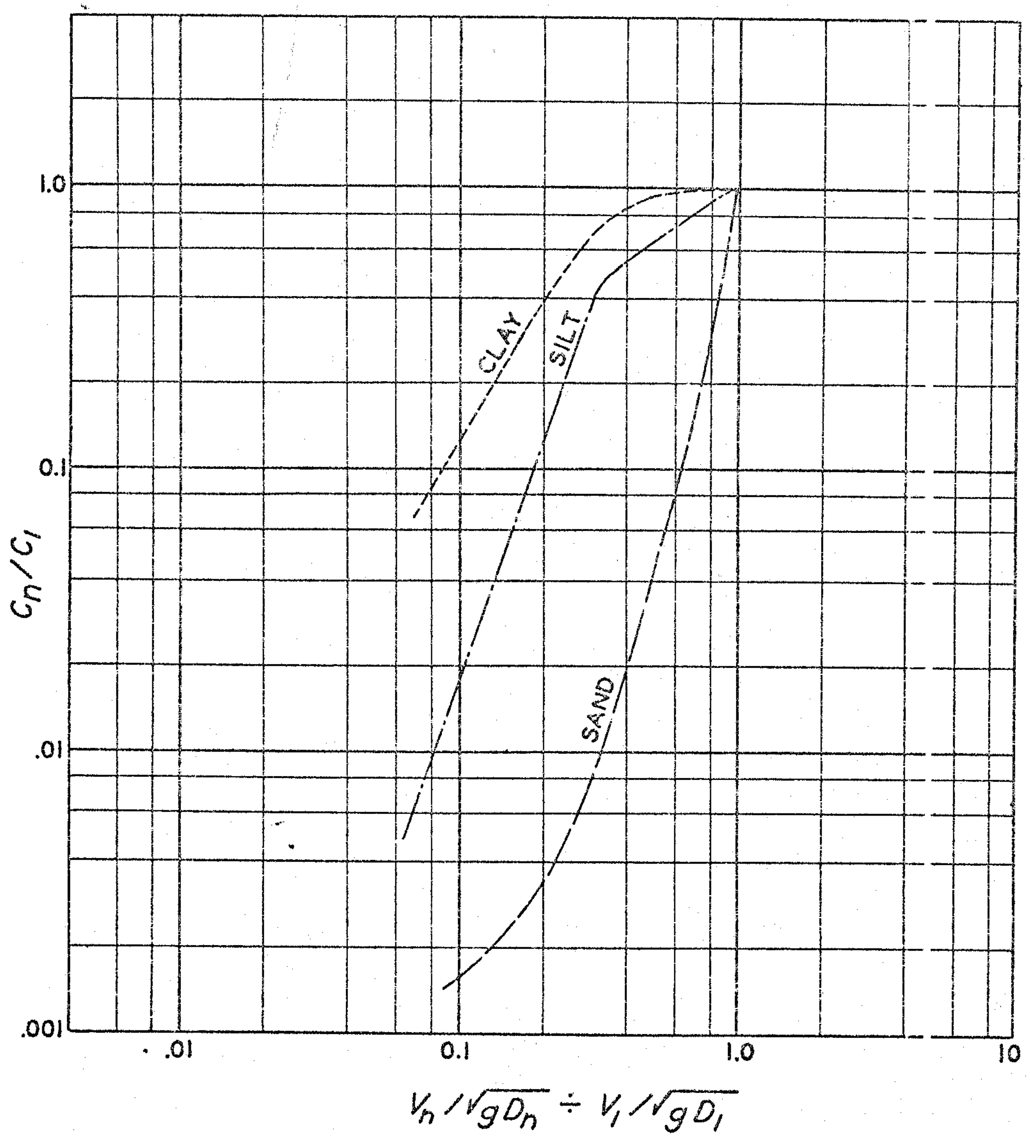


AVERAGE ANNUAL SEDIMENT
ACCUMULATION BY VOLUME,
LAKE DECATUR



15,000
10,000
9000
8000
7000
6000
5000
4000
3000
2000
1000
900
800
700
600
600
400
300
200
150
DISCHARGE AT OAKLEY, C.F.S.
30,000
20,000
10,000
TONS PER DAY
Measurement by State
May 4, 1936
SEDIMENT-DISCHARGE
RATING CURVE
TONS PER DAY
10,000
9000
8000
7000
6000
5000
4000
3000
2000
1000
900
800
700
600
500
400
300
200

PLATE 2



NOTES:
RATIO VALUES OF 1 AND 1 REPRESENT CONDITIONS IN OR NEAR THE NATURAL CHANNEL AT THE UPSTREAM END OF THE BACKWATER REACH AT THE HEAD OF THE RESERVOIR. RATIO VALUES LESS THAN 1 AND 1 REPRESENT CONDITIONS IN TRANSITION (BACKWATER REACH) BETWEEN THE NATURAL CHANNEL AND THE LEVEL POOL OF THE RESERVOIR.

C_n/C_1 IS THE RATIO OF THE SUSPENDED CONCENTRATION AT EACH RANGE THRU THE BACKWATER REACH TO THE SUSPENDED CONCENTRATION AT THE MOST UPSTREAM RANGE.

$V_n/\sqrt{gD_n} \div V_1/\sqrt{gD_1}$ IS THE RATIO OF $V/\sqrt{gD}$ AT EACH RANGE THRU THE BACKWATER REACH TO $V/\sqrt{gD}$ AT THE MOST UPSTREAM RANGE.

LEGEND

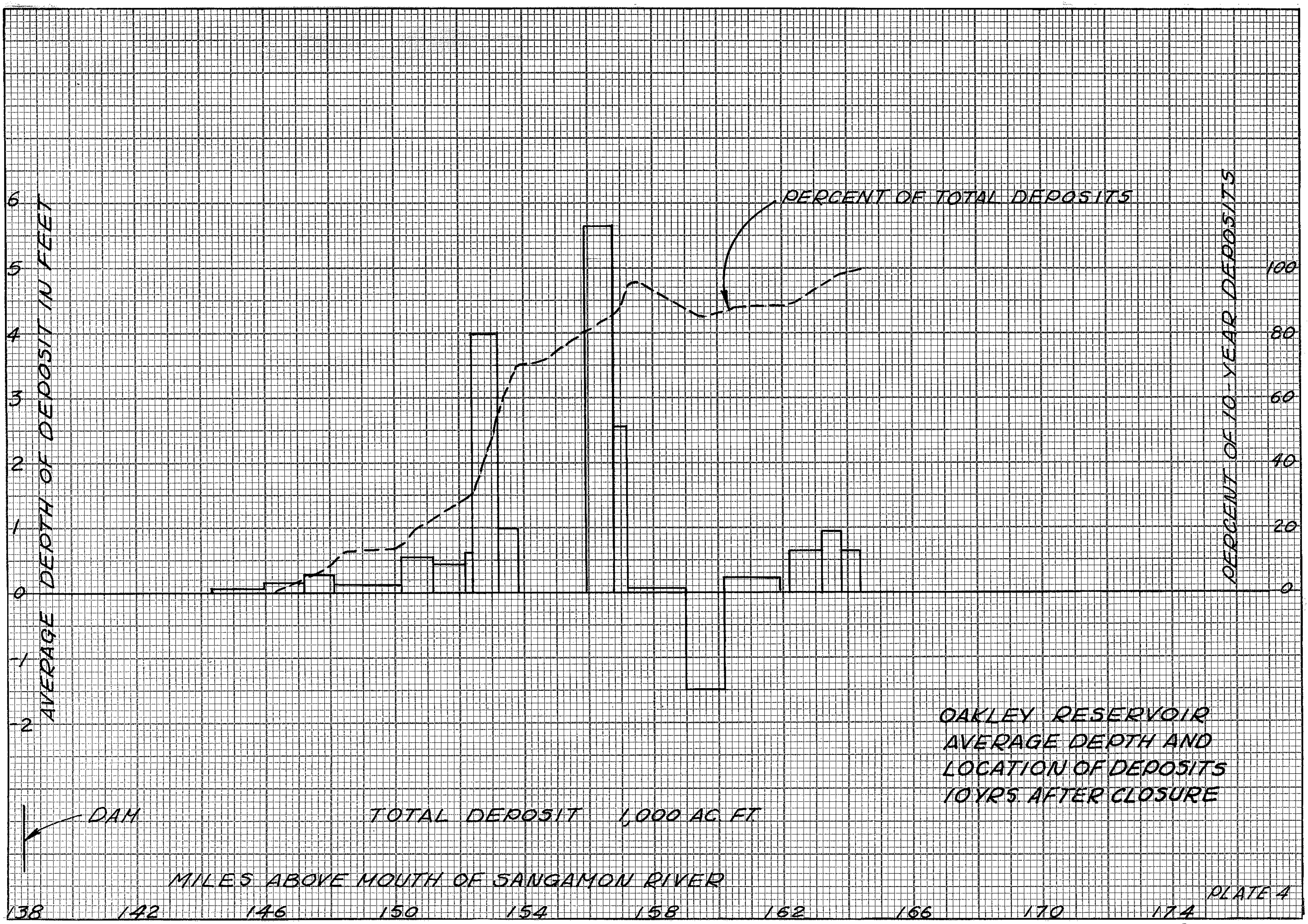
RESERVOIR	SURVEY DATE
DENISON	31 MAY 1946
"	16 SEPT 1946
"	23 SEPT 1946
"	14 MAY 1947
"	15 MAY 1947
FORT RANDALL	7-11 AUG 1953
FORT PECK	2-4 JUNE 1953
" "	9-10 JUNE 1953
" "	12 JUNE 1953
GARRISON	7-8 JULY 1954
AVERAGE	

RESERVOIR SEDIMENT STUDIES

C_n/C_1 and $V_n/\sqrt{gD_n} \div V_1/\sqrt{gD_1}$

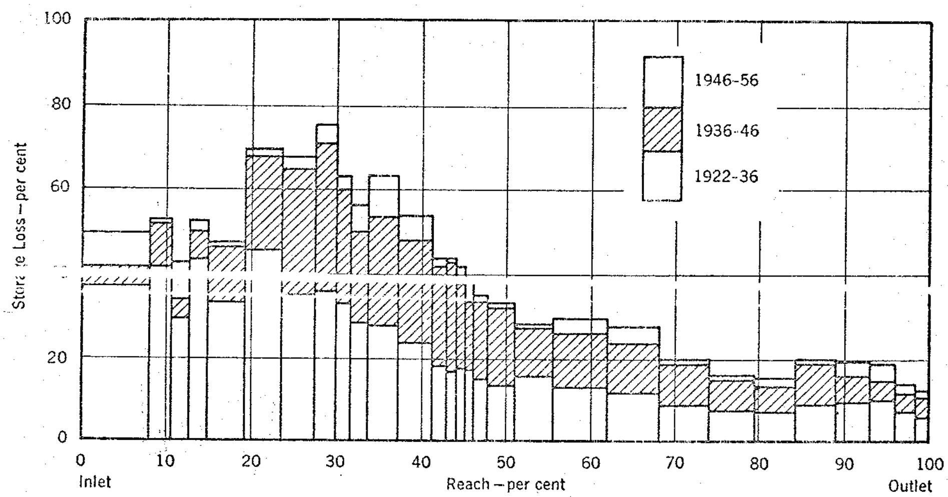
SAND - SILT - CLAY

THRU BACKWATER REACH



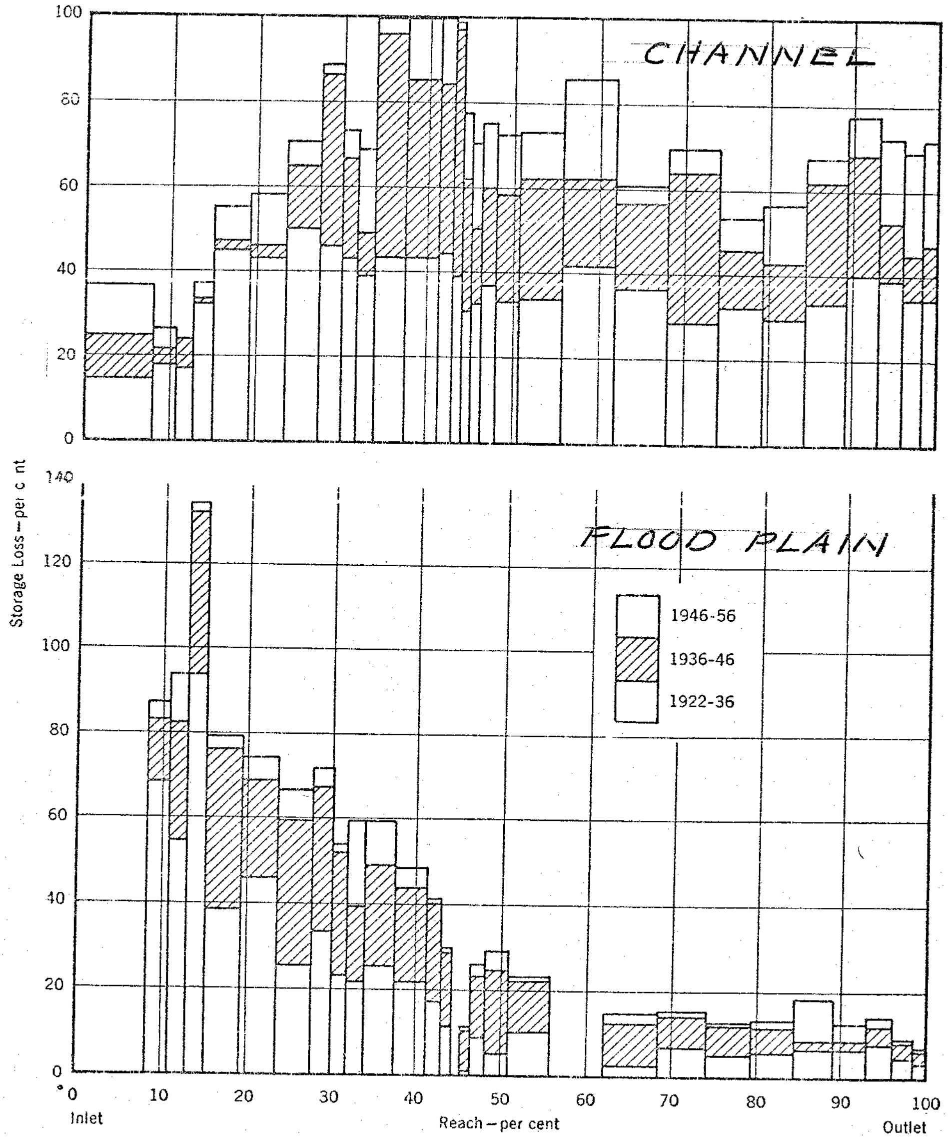
OAKLEY RESERVOIR
AVERAGE DEPTH AND
LOCATION OF DEPOSITS
10YRS. AFTER CLOSURE

PLATE 4

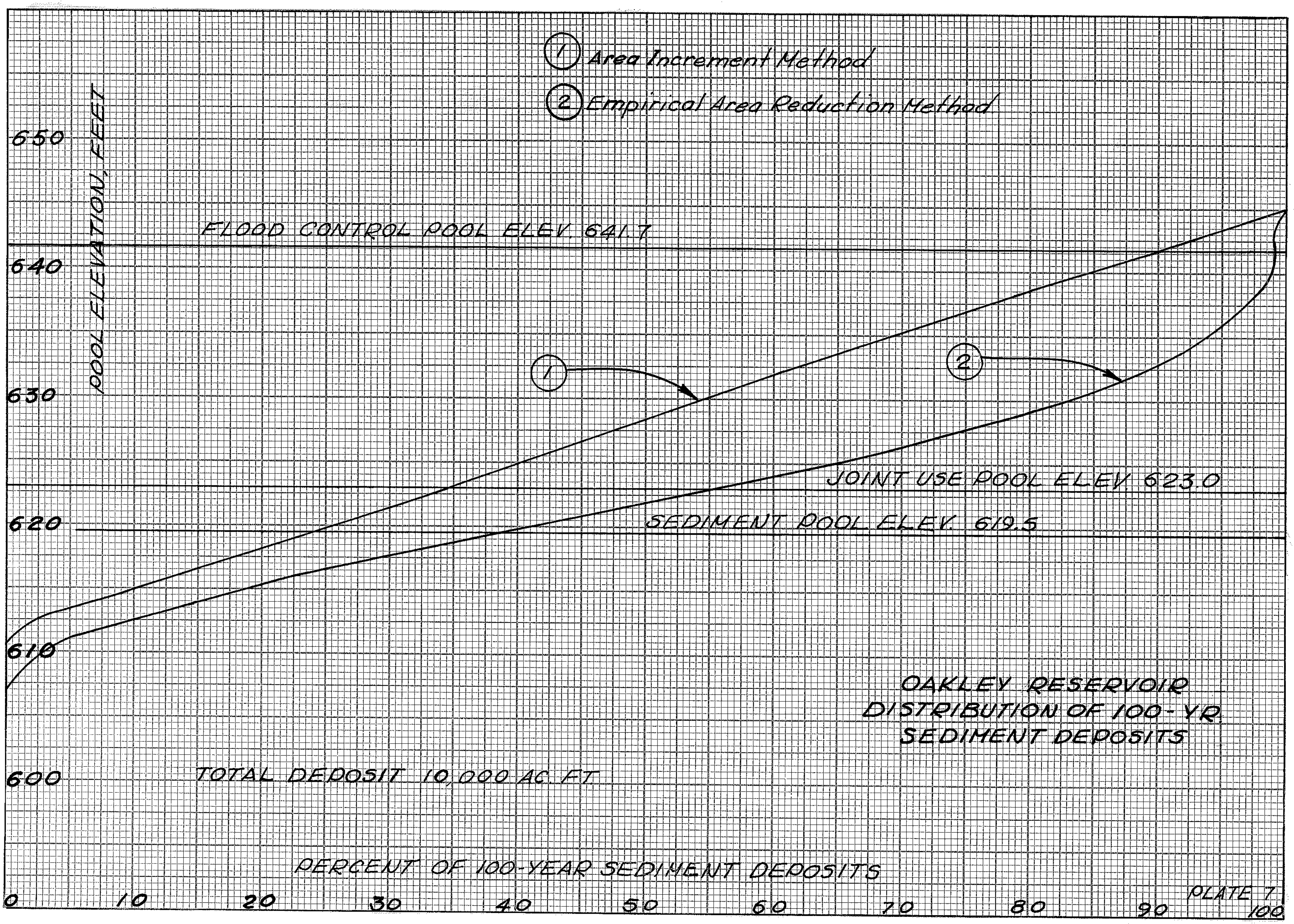


Distribution of Total Storage Loss in Lake Decatur

The largest depositions were during the first two periods. Most of the sediment collected near the inlet of the lake.



Storage Loss Distribution in Lake Decatur Channel and Flood Plain



OAKLEY RESERVOIR
DISTRIBUTION OF 100-YR.
SEDIMENT DEPOSITS

PLATE 7

MISSISSIPPI RIVER DAM No. 26
SEDIMENTATION STUDY

by

Fred B. Toffaleti[1]

INTRODUCTION

In a comprehensive study of Locks and Dam No. 26 on the Mississippi River at Alton, Illinois, it was concluded that major reconstruction was imminent. The locks were nearing a maximum capacity level of operation; were of outdated size for modern river traffic; and both locks and dam were badly in need of major repairs. In the consideration of alternatives to continuance of operations at the present site one plan provided for abandonment of the existing structures and constructing a dam at St. Louis conjunct with the existing Chain of Rocks Canal Locks. At this location the Missouri River would empty into the pool just upstream of the dam and at normal pool level, the low water plane at the confluence would be in the order of twenty feet higher than at present conditions. This then posed a question as to magnitude and extent of the sedimentary effects in the lower reaches of the Missouri River. This paper discusses the methods used for this determination and the results of the study.

Resolution

Fortunately, considerable data on characteristics of the lower Missouri River were available from a previous study involving shoaling in the St. Louis Harbor. Bed material samples had been obtained and fairly recent hydrographic surveys were made available by the Kansas City District. Also, the U. S. Geological Survey had submitted to the St. Louis District a report on the results of their suspended sediment sampling on the Missouri River at Hermann, Missouri.

Four items were necessary to initiate the study: Channel cross-sections; channel roughness coefficients; bed-material composition, and a sand discharge rating curve. The hydrographic survey maps provided data for plotting cross-sections and they also showed sufficient water-surface elevations and corresponding discharges from which channel roughness coefficients could be determined. A Mannings roughness coefficient of 0.020 was used for this study and while this is somewhat higher than indicated by the hydrographic surveys, it is slightly less than is understood to be used in design of river works in this vicinity. Data on bed material composition were available from a previous study. The other required item, a sand discharge rating curve, was constructed by a computational procedure. The method of computation used is contained in Technical Report No. 5 of the Corps of Engineers Committee on Channel Stabilization, "A Procedure for Computation of the Total River Sand Discharge and Detailed Distribution, Bed to Surface." A condensed version of this report is contained in Paper 6350, Journal of the Hydraulics Division, ASCE Proceedings, dated January 1969 and titled "Definitive Computations of Sand Discharge in Rivers."

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[1]Formerly Hydraulic Engineer, Lower Mississippi Valley Division

In the St. Louis Harbor shoaling study the Hydraulics Branch of the U. S. Army Engineer District, St. Louis, had constructed a sand discharge rating curve for the Missouri River reach, miles 95-103, by the above noted method. This rating curve, shown in Figure 1, was used for this study.

With these data a water-sand discharge relation was constructed as shown in Figure 2. For applicability to various channel widths the water and sand discharge are for a one-foot width. The plot also shows the water-surface slope and the depth of the section that will produce a given water and sand discharge. The water discharge graphs are readily computed by slide rule and desk calculator by the solution of the expression

$$R = \frac{0.075Q^{3/5}}{S^{0.30}} \qquad \text{(for n=0.020)}$$

in which R is the section depth, Q the water discharge in cfs and S the water-surface slope.

The fixing of sand discharge graphs will require use of an electronic computer and a double plot. Required data for the computation are mean velocity of the section, depth of the section, water temperature, width of section, D_{65} of the bed material, the water surface or energy slope, composition of the bed material, and the grain fall velocity. The water temperature may be a selected value, say 55°F, and from the plot of water discharge versus slope and depth, the sand discharge computations are made along each depth line at each intersecting water-discharge line. By this selection of points for computation, all necessary data for a plot of sand discharge versus slope for the given depths are provided. From this initial plot the desired detail for plotting sand discharge versus depth and slope can be extracted for plotting on the chart showing the water discharge versus depth and slope. The combined plot provides a water-sand discharge relation applicable as a stabilized condition of depth and slope for any water and sand discharge rates.

The usefulness of this chart is readily apparent as it can be used to plot a backwater curve and also show the mean depth of a stable channel for the conditions posed.

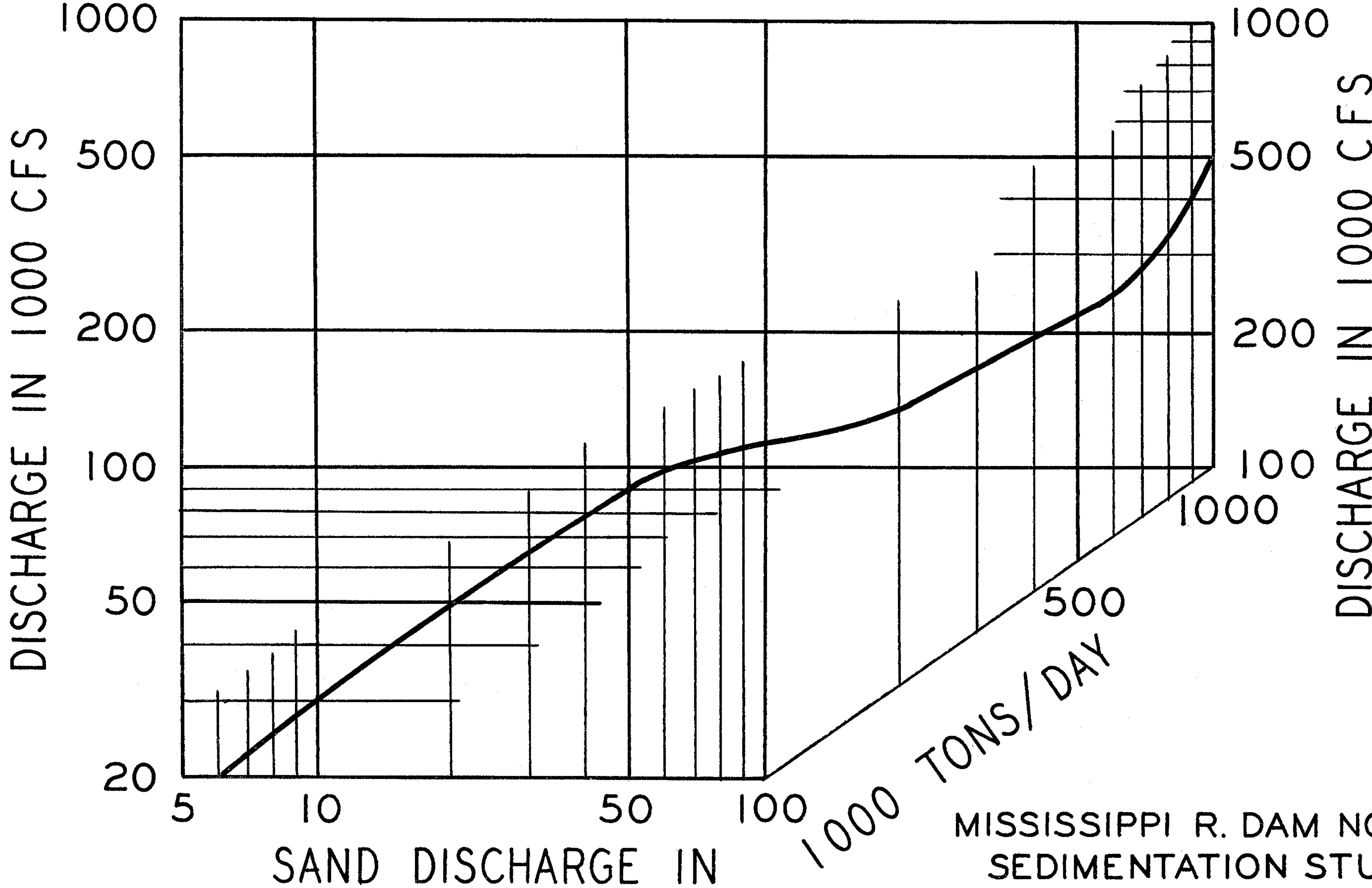
Results

For the purposes of this paper the results shown are those to be expected for a Missouri River flow of 100,000 cfs. Figure 3 shows profiles of the existing mean bottom and the water-surface profile under these conditions for the 100,000 cfs flow, and at initial pool assuming existing bottom elevations to prevail at that time. This was determined by ordinary backwater computations and indicates a raised profile at initial pool that extends about 26 miles upstream. In the determination of a future water-surface profile and the extent of channel filling that was to be expected it was indicated by the water-sand discharge relation that the existing channel stabilization works would, in time, be practically completely covered with sand. It was concluded that in this evolution to an unrestricted channel it would lose its present identity as a navigable channel. Thus, the funding of construction of a dam at St. Louis should include the cost of rehabilitating the navigable channel on the lower Missouri River. On this basis, water surface and mean-bottom profiles were developed with the existing channel works considered reconstructed in relation to indicated future developments the same as in existing conditions. Figure 3 shows these future water surface and mean-bottom profiles for a flow of 100,000 cfs as developed by use of the water-sand discharge relations shown in Figure 2. It is to be noted that backwater effects now extend about 37 miles upstream.

Profiles for flows other than 100,000 cfs were determined; however, for the purpose of this paper, only this flow was selected for illustration. A typical example of other flows in a cross-section is shown in Figure 4. This is considered an excellent depiction of the efficacy of the reconstructed works for passing water and sand over a rather wide range of flow with no significant change in mean-bottom elevation. The indication is that a flood rise to 200,000 cfs after a long period of flow at about 50,000 cfs would first tend to degrade the channel and then aggrade back to the low-flow level. The indicated stable mean-bottom elevation for 100,000 cfs is four feet below that of 50,000 cfs, that of 150,000 cfs only one foot below and for 200,000 cfs the same as for 50,000 cfs.

Conclusions

No attempt was made in this study to develop a sequential time evolution from present to future conditions. This was not necessary as the ultimate condition was to be the basis for estimating rehabilitation and mitigation costs. However, complete channel filling should occur rather quickly, say easily within ten years. Initial channel filling would occur in the vicinity of mile 25, and extension would be both upstream and downstream from this point with the more rapid growth in the downstream direction. The ultimate extension of pool effects up to about 40 miles upstream does not seem improbable and the results shown seem within the limit of expectancy for a raising of the low water about 20 feet at the mouth of the Missouri River.

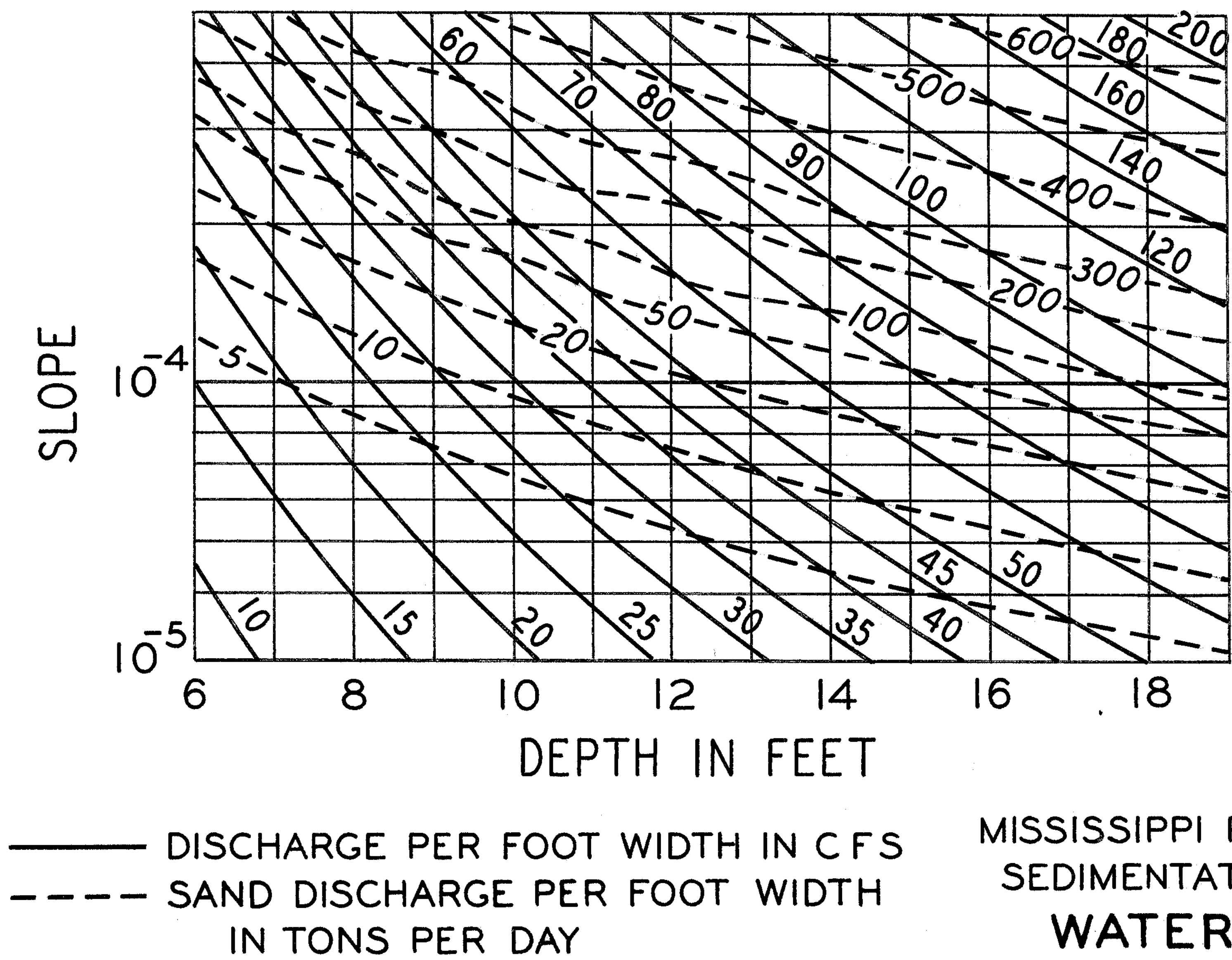


MISSISSIPPI R. DAM NO. 26
SEDIMENTATION STUDY

SAND DISCHARGE
RATING CURVE

MISSOURI RIVER, MILE 95–103

FIG. 1

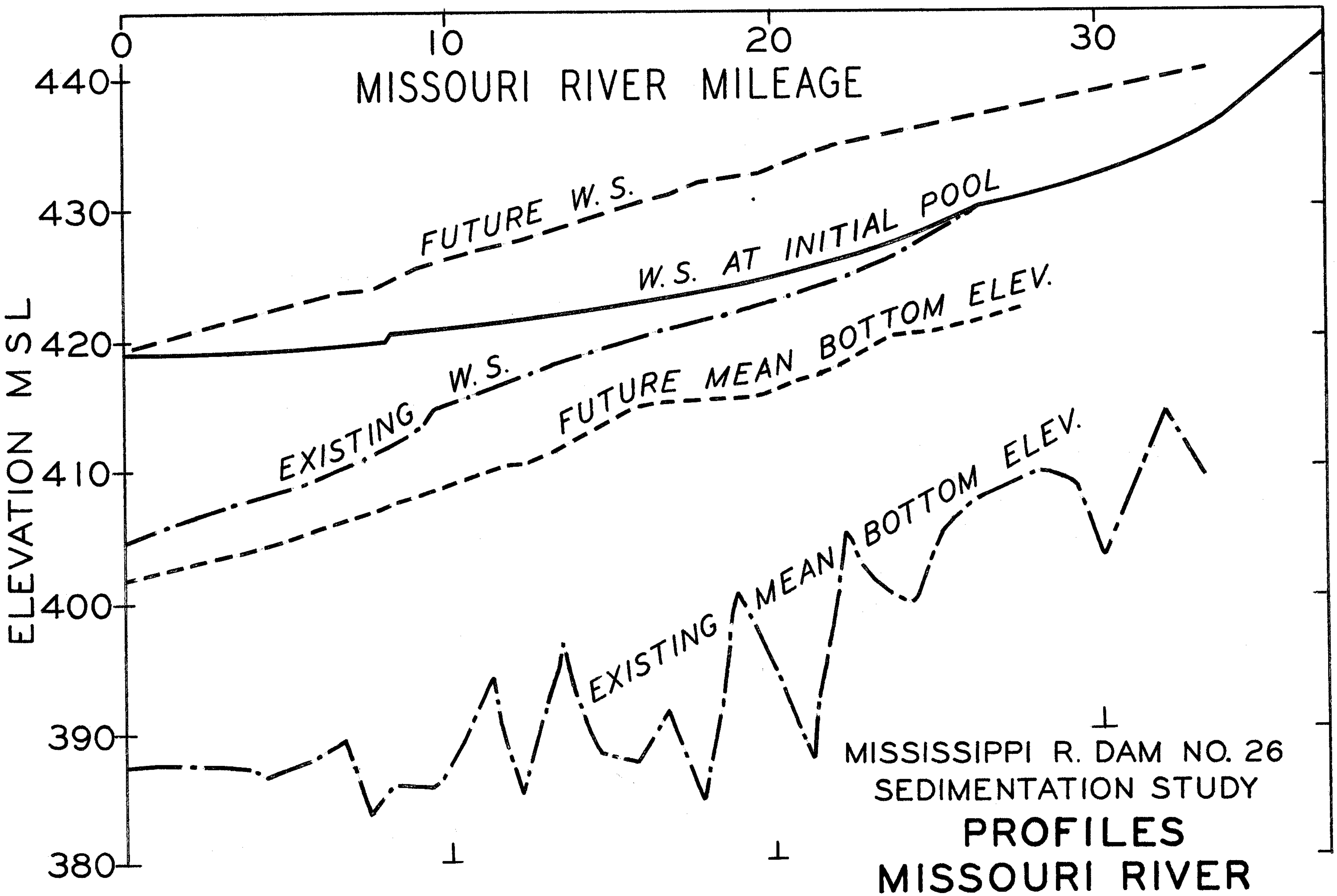


MISSISSIPPI R. DAM NO. 26
SEDIMENTATION STUDY

WATER- SAND DISCHARGE RELATION

$n = 0.020$

FIG. 2

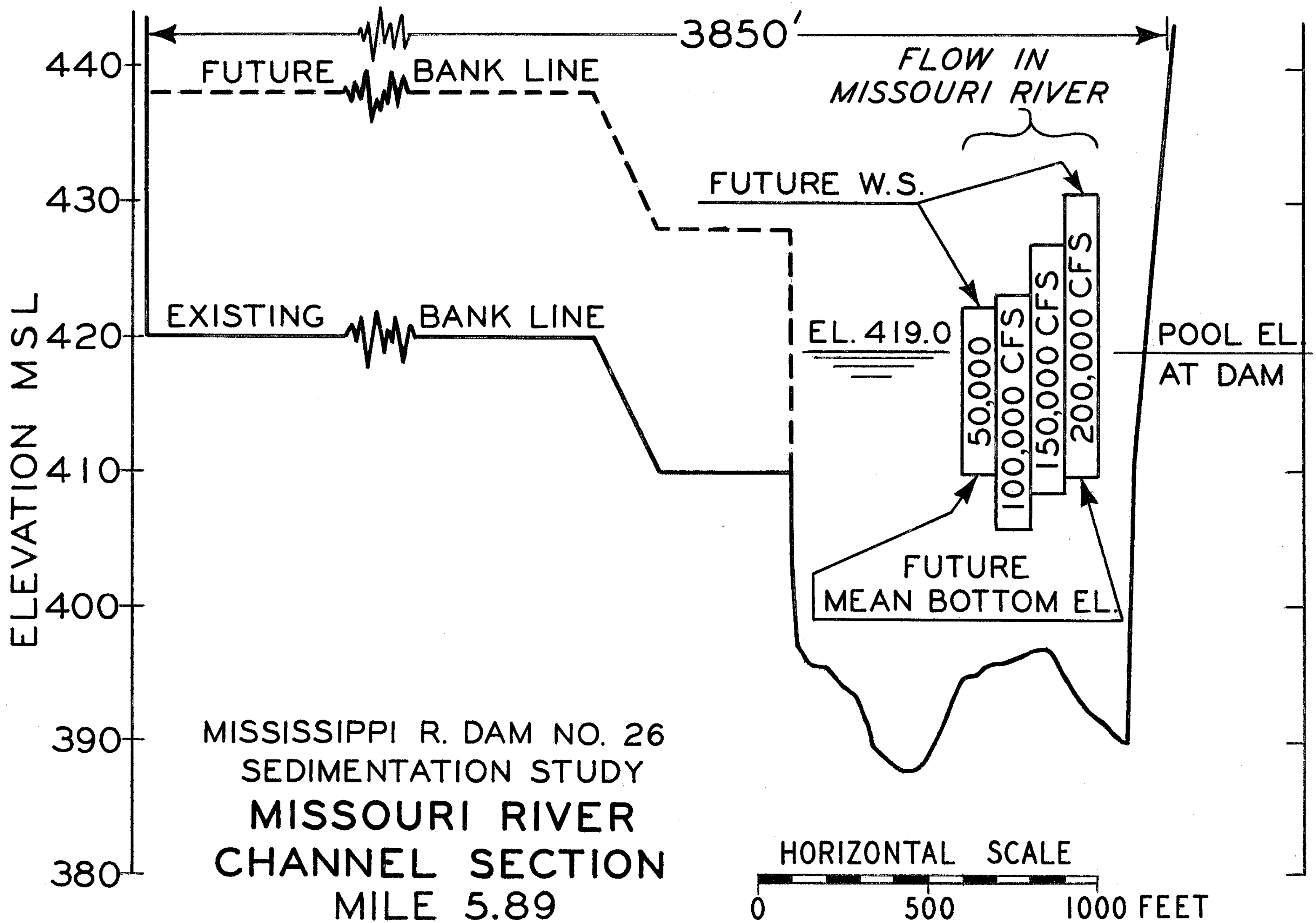


MISSISSIPPI R. DAM NO. 26
SEDIMENTATION STUDY

PROFILES
MISSOURI RIVER

DISCHARGE: 100,000 CFS

FIG. 3



MISSISSIPPI R. DAM NO. 26
SEDIMENTATION STUDY

MISSOURI RIVER
CHANNEL SECTION
MILE 5.89

FIG. 4

SEDIMENT PROBLEMS
IN
ST. LOUIS HARBOR

by

James R. Tuttle[1]

INTRODUCTION

The reach of Mississippi River known as the St. Louis Harbor begins at the mouth of the River Des Peres (Mile 172), and ends at the mouth of Watkins Creek (Mile 191.2), representing a distance of 19.2 miles. However, for the purposes of this paper, the lower limit is extended downstream to Jefferson Barracks Bridge (Mile 169) and the upper limit terminates at the entrance to Chain of Rocks Canal (Mile 184). This gives full consideration to all piers and docks in the vicinity and includes the problem areas of interest. (See Exhibits 1 and 1-A.)

The Missouri River is confluent with the Mississippi River at about river mile 195, 11 miles above the upstream limit of the harbor, and constitutes, on the average, about 45 percent of the total flow at St. Louis.

Normally, the authorized nine-foot navigation depth is available even at low stages; however, there have been times when less than project depth has occurred at the dock and terminals. This condition prompted local interest to petition the Corps of Engineers for aid in determining the causes and investigating means of eliminating or reducing the problem.

Suspended sediment samples have been collected by the USGS from the Missouri River at Hermann, Missouri and from the Mississippi River at St. Louis since 1948, a period of 22 years. Sand discharge rating curves developed from these measurements indicate that the Mississippi River at St. Louis is not capable of carrying as much sand as the Missouri River. Hydrographic surveys for the period 1946-1963 indicate a net deposition of about 21 million tons of sand in the harbor area. This tends to support the above rating curves; however, inadequate navigation depths in the harbor area have occurred infrequently, leading one to suspect that the period in which the hydrographic surveys were conducted covered a portion of a cycle in which a balance of scour and deposition occurred.

A moveable bed hydraulic model is being operated by Waterways Experiment Station to investigate general patterns of sediment movement through the harbor area. It will be used to test the effectiveness of various remedial measures such as: (1)Vane dikes; (2) longitudinal dike with

1 Chief, Hydraulic and Reservoir Regulation Section, Lower Mississippi Valley Division

fill behind to extend the west bank riverward; and (3) L-shaped dikes.

The purpose of this paper is four-fold: (1) To describe the problem in St. Louis Harbor; (2) to describe the water and sediment discharge characteristics of the Missouri and Mississippi River; (3) to describe some proposed remedial measures being investigated; (4) and to compare different methods used in sediment transport analysis. The mechanics of each method will not be discussed, only a comparison of rating curves developed at St. Louis by each method.

General Features of Harbor Area

St. Louis Harbor proper includes both the Missouri and Illinois banks of the Mississippi River in the 15-mile reach. The harbor contains some 50 docks and terminals, 35 of which are on the Missouri shore. Flow is confined between artificially stabilized banks and levees. At normal stages the channel is nearly uniform in width and slightly curved through the harbor reach. The width of flow varies from about 1500 feet at low stages to about 1800 feet at bankfull stage, going to about 2800 feet when overbank. Some meandering takes place within the channel confines. There are six existing bridges in the harbor with one new bridge and another being planned.

Existing Projects

The plan for regulation of the middle Mississippi River provides for continuous improvement working downstream from St. Louis, utilizing revetment and permeable kides to reduce the width of the river to 2,500 feet. The existing project for the middle Mississippi River provides for obtaining and maintaining a minimum channel width of not less than 300 feet at low water, with additional widths in bends from the mouth of the Ohio River to the northern boundary of St. Louis, to be obtained by regulating works and dredging.

Chain of Rocks Canal at the upper extremity of the harbor, provides for bypassing the hazardous Chain of Rocks reach of the Mississippi River. The upper terminus of the canal is at mile 194, about one mile below the mouth of the Missouri River.

Dam No. 27 is an existing rockfilled structure located in the Mississippi River channel at river mile 190.3. The dam, completed in 1963, consists of a broad-crested weir with a length of 2,140 feet at crest elevation 395 in combination with a notch of about 676 feet at crest elevation 391. The purpose of the dam was to raise tailwater at Lock and Dam No. 26 upstream of the confluence of the Missouri and Mississippi Rivers.

Discharge Characteristics

The minimum flow of record is 18,000 cfs which occurred in December 1863. The maximum stage of record modified for present conditions is 52 feet at St. Louis and it is estimated that the peak discharge was 1,300,000 cfs. The mean flow for the period 1946-67 is 166,700 cfs at St. Louis on the Mississippi River and 72,200 cfs at Hermann on the Missouri River. During this period the percent contribution by the Missouri River ranged between 33.4 in 1954 to 54.4 in 1949. The percentage figures are based on annual mean flows and are shown in Table I. The flow contribution of the Missouri River was above average in ten of the 22 years shown. (See Tabel 1.)

Problem in the St. Louis Harbor

Actually there is no record of the exact times during which docks and terminals were inoperative due to inadequate depths. Local interest indicates that the most critical period occurred during 1963-64 which were very low flow years. However, it is indicated that other periods have occurred where serious interferences with operation of port facilities were caused by inadequate depths. The problem then becomes one of trying to determine the causes of sedimentation in the harbor and vicinity and to find the most feasible means of reducing, eliminating, or altering the causes.

Suspended Sediment Discharge and Sediment Size

Data on yearly suspended sediment discharges are rather irregular, but the totals for 1949-63 indicate an average decrease from Hermann and Hannibal to St. Louis of about nine percent. The largest percentage decrease from Hermann and Hannibal to St. Louis for one year was 29 percent in 1958. In the 15-year period above, there was an increase from Hermann and Hannibal to St. Louis in six years and a decrease in nine years, with the average suspended sediment discharge at Hannibal being about 13 percent of that at St. Louis. The volume of deposition arrived at by analyzing the 15-year record amounts to about 232 million tons. To illustrate this amount of sediment loss, if it were deposited uniformly on the riverbed between Hermann, Hannibal, and St. Louis, it would form a deposit 2.9 feet thick over the entire 228-mile reach. At the other extreme, if a 25-mile reach were considered, it would form a deposit 26 feet thick. Another consideration is the measurements themselves; using average annual figures only, it was found that by increasing the average annual suspended sediment expressed in parts per million (PPM) at St. Louis by 15 percent would eliminate the 232 million ton deficiency.

TABLE 1

Annual mean flow at Hermann and St. Louis and percent of Hermann flow St. Louis

Year (1)	Mean Flow in 1,000 CFS Hermann (2)	St. Louis (3)	% of Flow (2) ÷ (3) (4)
1946	68.2	184.0	37.1
1947	102	221.6	46.0
1948	79.6	158.5	50.2
1949	95.9	176.2	54.4
1950	91.0	194.9	46.7
1951	150	291.7	51.4
1952	88.7	204.2	43.4
1953	54.5	139.3	39.1
1954	42.0	125.7	33.4
1955	46.4	119.8	38.7
1956	32.6	89.6	36.4
1957	50.7	131.5	38.6
1958	79.6	141.7	50.2
1959	61.0	152.1	40.0
1960	74.8	195.5	38.3
1961	92.1	195.8	47.0
1962	73.2	191.5	38.3
1963	42.0	104.4	40.0
1964	48.6	102.5	47.5
1965	86.7	223.1	38.7
1966	53.7	151.1	35.5
1967	75.8	173.0	44.0
AVE	72.2	166.7	43.3

Care should be exercised when trying to draw conclusions from small percentage differences in sediment discharge.

Approximately 75 percent of the suspended sediment is finer than 0.062 mm in the harbor area.

Bed Material

The size distribution is variable with location in cross section. Near the banks the material is mostly fine sand. It is also variable with time. Bed material samples taken in 1951-1952 indicate median diameters that range from 0.6 to 1.1 mm while measurements in 1953-1956 indicate median diameters that averaged about 0.3 mm in size. The years 1951-52 were high-flow years, while 1953-56 were low-flow years with the Missouri River contribution of flow above average for the high-flow years and below average for the low-flow years. Jordan[1] found that a fair relation was obtained when average median diameters were plotted against mean discharge for one-and two-year periods, but was not good for periods of lesser duration. This indicates that the bed material size is influenced more by the depth of scour than by the selective removal or deposition of fine and medium sands from the upper few millimeters of the bed during short periods of high or low flow.

Sand Discharges

Yearly sand discharges were calculated from the average percentage sand in the available size analysis for each year. The relationship of yearly sand discharge to streamflow is more consistent than daily sand discharges calculated for days of available particle size analyses plotted against the corresponding daily streamflow. An average curve thus developed (1949-63) was used to compute daily sand discharges, which were then added to give the yearly totals that were used in subsequent calculations.

The Mississippi River at Hannibal carries very little sand in the suspended sediment. The average of all particle size analyses from 1951 through 1962 showed only two percent sand. Subsequently, all calculations of sand discharge at Hannibal were based on two percent sand.
At Hermann the average annual flow for the period 1946-1967 is 72,200 cfs carrying a sand load of approximately 30 million tons per year. At St. Louis the average annual flow for the same period is 166,700 cfs which carries about 22 million tons of sand per year. See Figures 1 and 2.

Aggradation and Degradation

A comparison of sand discharges indicates an average deficiency of sediment transport capability at St. Louis of eight million tons per year. Naturally, if this was the case, the harbor would not exist at the present date; therefore, a mere comparison of rating curves, which in themselves are only useful as indications of sand discharges, is not adequate. Applied to the period 1949-1963 the annual totals show a net deposition (Hermann plus Hannibal minus St. Louis) of 95 million tons. Subtotals for 1949-1958 show substantial deposition but the subtotals for 1959-1963 show very little difference in sand loads. These results are in substantial agreement with results of hydrographic surveys in the harbor area which showed about 22 million tons deposition in the harbor from 1946 to 1959 and little net change (net scour of about one million tons) from 1959 to 1964. This agreement indicates that the sediment deposition in the harbor area involves more than merely the local flow conditions at the docks, but is probably a part of a larger pattern of deposition which is related to the inflow from the Missouri River and the sediment-carrying capacity of the Mississippi River.

Data furnished by St. Louis District provide another view of what may be happening in the harbor area. Fig. 3 is an accumulative plot of the amount of sediment transported past Hermann that did not pass Reach 1 in St. Louis Harbor. It is realized that this plot contains the same errors as are present in the rating curves used to arrive at the plotted points; however, the plot indicates the periods in which scour or deposition can reasonably be expected if the rating curves and other data used to this point are in the right ball park. It is regrettable that the hydrographic survey data were not broken down in more time detail. The hydrographic survey data were given as being from 1946-1959 and indicated net deposition for the period, while Figure 3 indicates that the deposition took place in 1948, 1949, 1950, and 1951 and some scour occurred from 1952 to 1957, then deposition in 1958. An examination of the distribution of average monthly flows reduced to yearly percentages indicates that the deposition years were years in which Missouri River flow contribution was above average and the years of scour were below average years of contribution. This is in agreement with the thought that deposition will take place with high Missouri River flows and low Mississippi River flows. Further light can be shown from hydrographic surveys conducted above the Chain of Rocks reach of the Mississippi River. Table 2 shows the ranges surveyed and the results. Also, notice that the trend indicated by Figure 3 somewhat agrees with the hydrographic survey where little change took place between 1959 and 1964 if the sharp recession shown for the first part of 1960 was moved back into 1959.

Comparison of Different Computation Methods

The comparison of various methods available for use in computing bed-material discharges is not intended to expose any one method as being better or more exact than the other. The purpose is to examine each in relation to the measured bed-material discharge to see which method is the more desirable for the particular location. In this case Toffaleti's method compares the more favorably and would seem to be the most desirable method for use at St. Louis. The location at Hermann was not analyzed; therefore, no conclusion can be drawn as to the best method at this location. As in most cases involving sediment transport, care should be exercised when attempting to substitute computational methods for actual measurements and detailed data. With the same token, the computational method giving the best results at one location is not necessarily the best at some distant location. See Figure 4 for a plot of the different methods as related to the measured bed-material discharge. Table 3 shows the measurements used to obtain the points plotted.

Engineering Studies

Many proposals were discussed in searching for possible means of reducing or eliminating the problems in St. Louis Harbor. They were:

a. The result of reducing harbor widths with resultant accelerations of flows. This might produce a self-cleaning prism; however, the dock owner would have to add facilities riverward to continue providing loading and unloading capability.

b. Consider the effectiveness and feasibility of obtaining supplemental flows to reduce the frequency of sudden decreases in natural flows. At the outset, this appears to be a costly solution.

c. Investigate a bifurcated channel with a channel on the Missouri and Illinois sides, respectively. Flow distribution and therefore sediment distribution would be a problem and would surely require excessive model studies. Also, cross channel openings for small craft would be necessary, but would create navigation hazards.

d. Consider dredging as a solution. Disposal of spoil becomes a problem. The banks could not be used nor the navigation channel.

e. The feasibility of constructing a navigation dam downstream of the harbor to provide greater depths at low water. Aside from being a costly item, the elevation of a slack water pool would be limited by the elevation of gravity drainage outlets in nearby flood protection works and in River Des Peres and Prairie du Point Creek.

f. Model studies. It was concluded that model studies were definitely desirable especially in light of the uncertainty of the numerical analyses. Since the model studies are in progress, they will not be discussed in this paper. Possible solutions to be tested were concluded to be (1) vane dikes, (2) longitudinal dikes with fill behind to extend the Missouri bank riverward, and (3) L-shaped dikes. Additional measures and/or tests will be conducted as necessary during the testing program.

Conclusions

The problem of inadequate depths at docks within the St. Louis Harbor apparently occurs infrequently but is quite troublesome when they do occur. The period 1963-1964 was one of unusually low water and produced the most critical depths experienced.

Comparison of suspended sediment at Hermann and St. Louis indicate large losses when compared on the basis of a period of record. The average annual suspended sediment discharges are 176.4 and 160.9 million tons at Hermann and St. Louis, respectively.

During high flow years bed-material samples indicate median diameters ranging from 0.6 to 1.1 mm as compared to median diameters of about 0.3 mm during years of low flow.

Sand rating curves developed at Hermann and St. Louis indicate that the Mississippi River at St. Louis is not capable of carrying the sand load delivered by the Missouri River. The respective annual ratings are, 30 million tons at Hermann and 22 million tons at St. Louis.

From totals of sand transported, hydrographic surveys, and examination of annual flows at Hermann and St. Louis, it is concluded that the sediment deposition and/or scour in the harbor area involves more than merely the local flow conditions at the docks, though they are also important, but is probably a part of a larger pattern of deposition which is related to the inflow from the Missouri River and the sediment-carrying capacity of the Mississippi River.

The comparison of various methods available for use in computing bed-material discharges indicates that Toffaleti's method compares more favorably with measured data in St. Louis Harbor.

Data are not available in sufficient detail to make a reasonable determination of causes with a numerical analyses; therefore, a model study is being conducted by the Waterways Experiment Station on the St. Louis Harbor.

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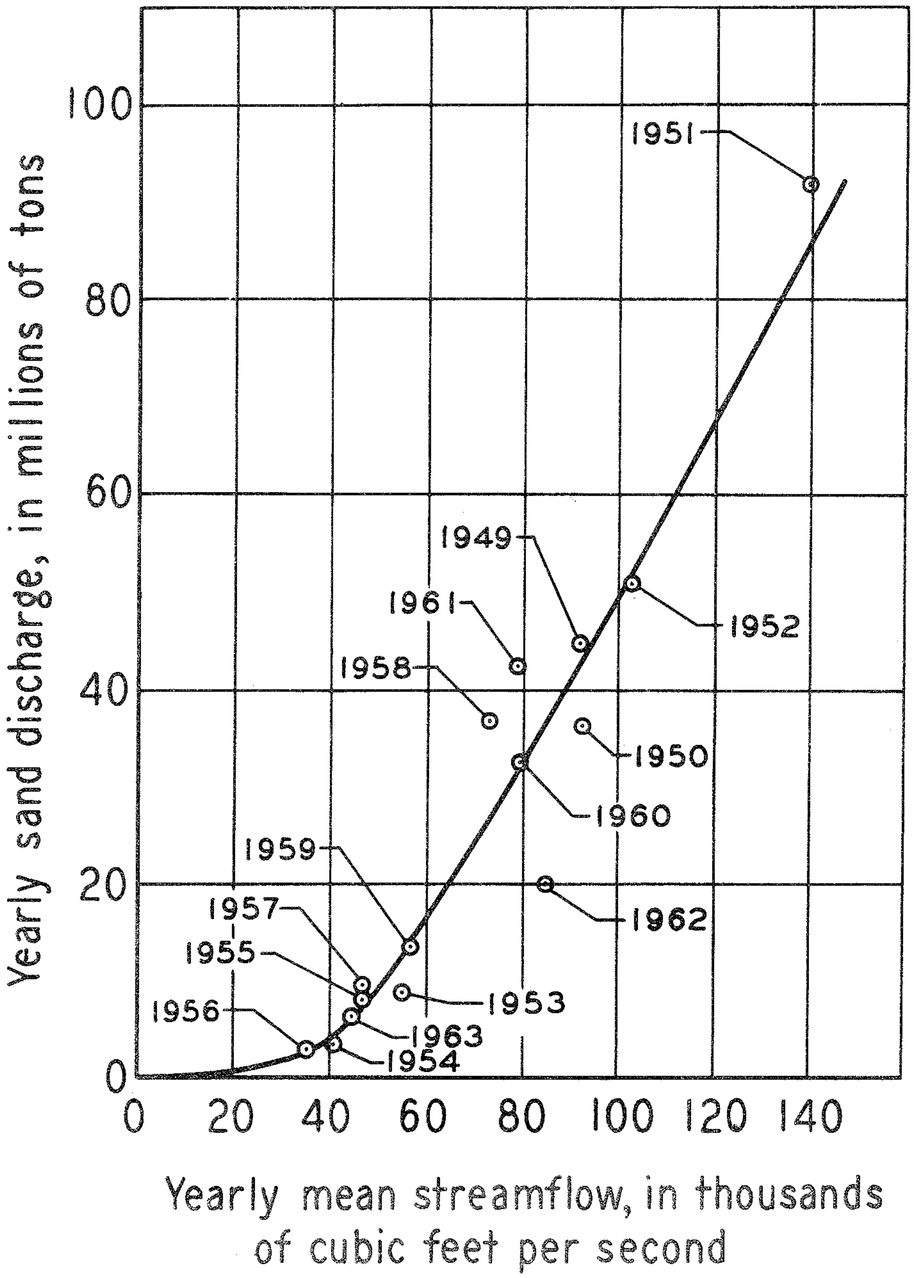


FIG. 1-RELATION OF YEARLY SAND DISCHARGE TO STREAMFLOW, MISSOURI RIVER AT HERMANN, MISSOURI

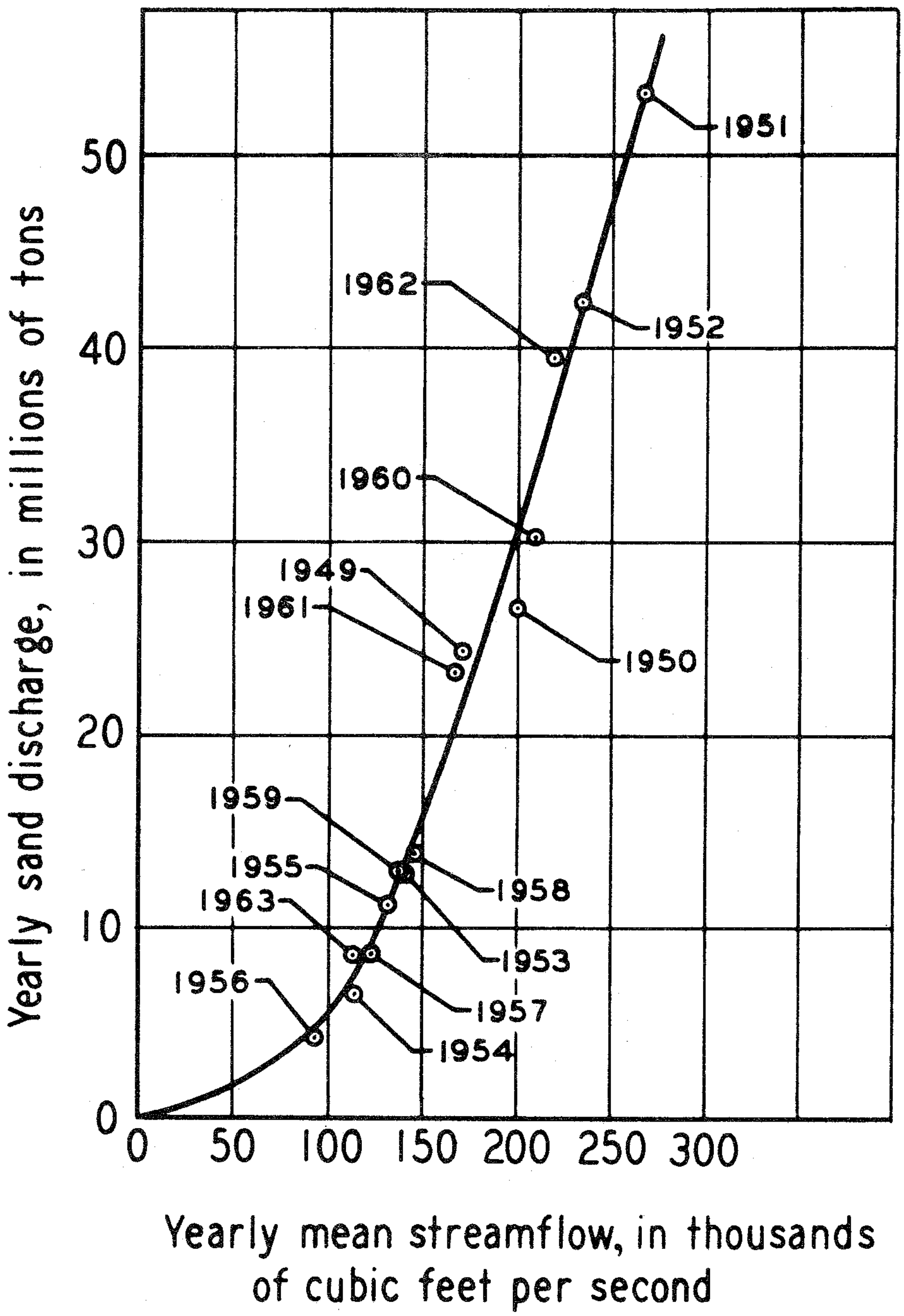
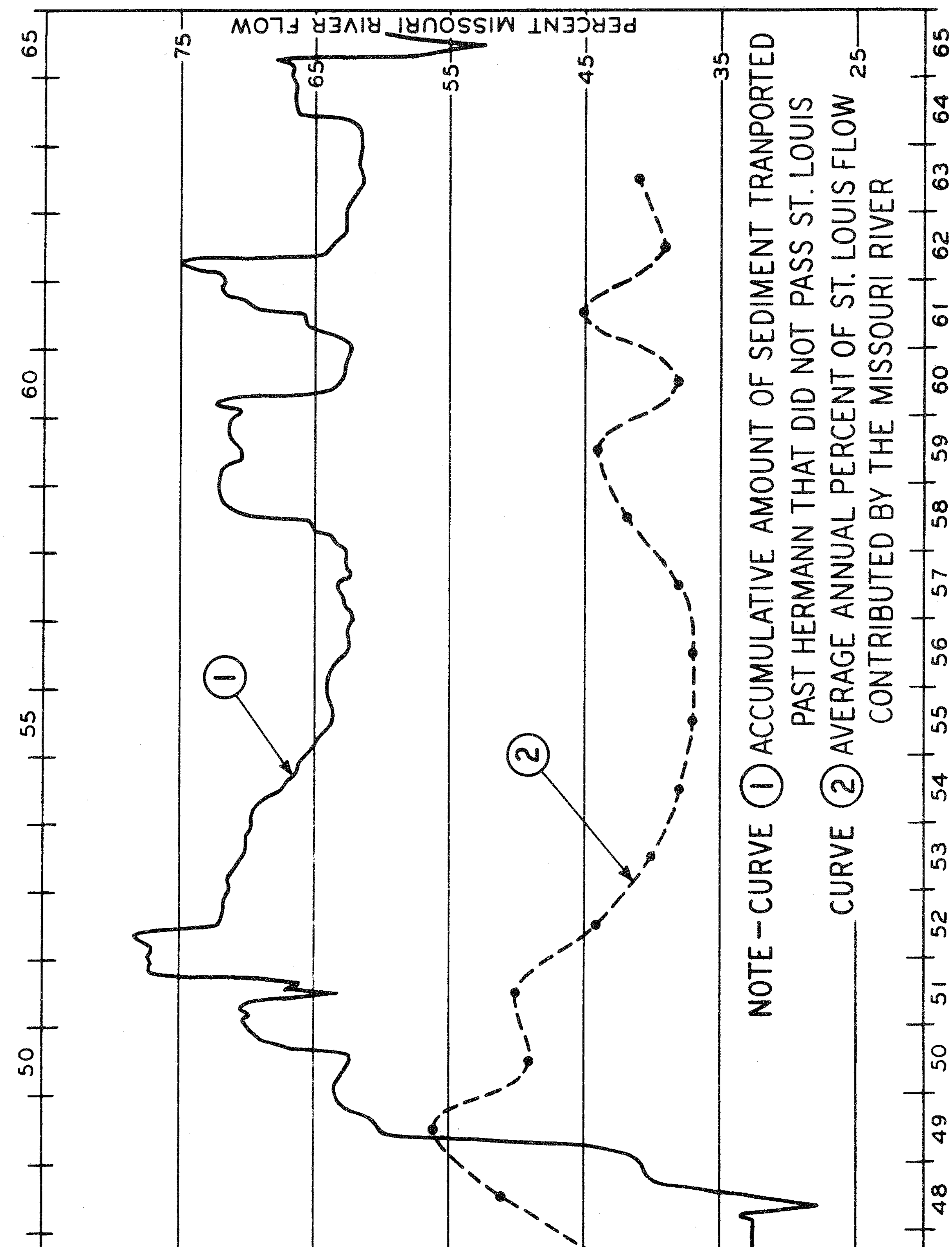


FIG. 2-RELATION OF YEARLY SAND DISCHARGE TO STREAMFLOW, MISSISSIPPI RIVER AT ST. LOUIS, MISSOURI



SAND DISCHARGE IN 1,000,000 TONS
PERCENT MISSOURI RIVER FLOW
+30
+20
+10
0
-10
-20
-30
75
65
55
45
35
25
46
47
48
49
50
51
52
53
54
55
56
57
58
59
60
61
62
63
64
65
YEAR
NOTE — CURVE (1) ACCUMULATIVE AMOUNT OF SEDIMENT TRANPORTED PAST HERMANN THAT DID NOT PASS ST. LOUIS
CURVE (2) AVERAGE ANNUAL PERCENT OF ST. LOUIS FLOW CONTRIBUTED BY THE MISSOURI RIVER

FIG. 3

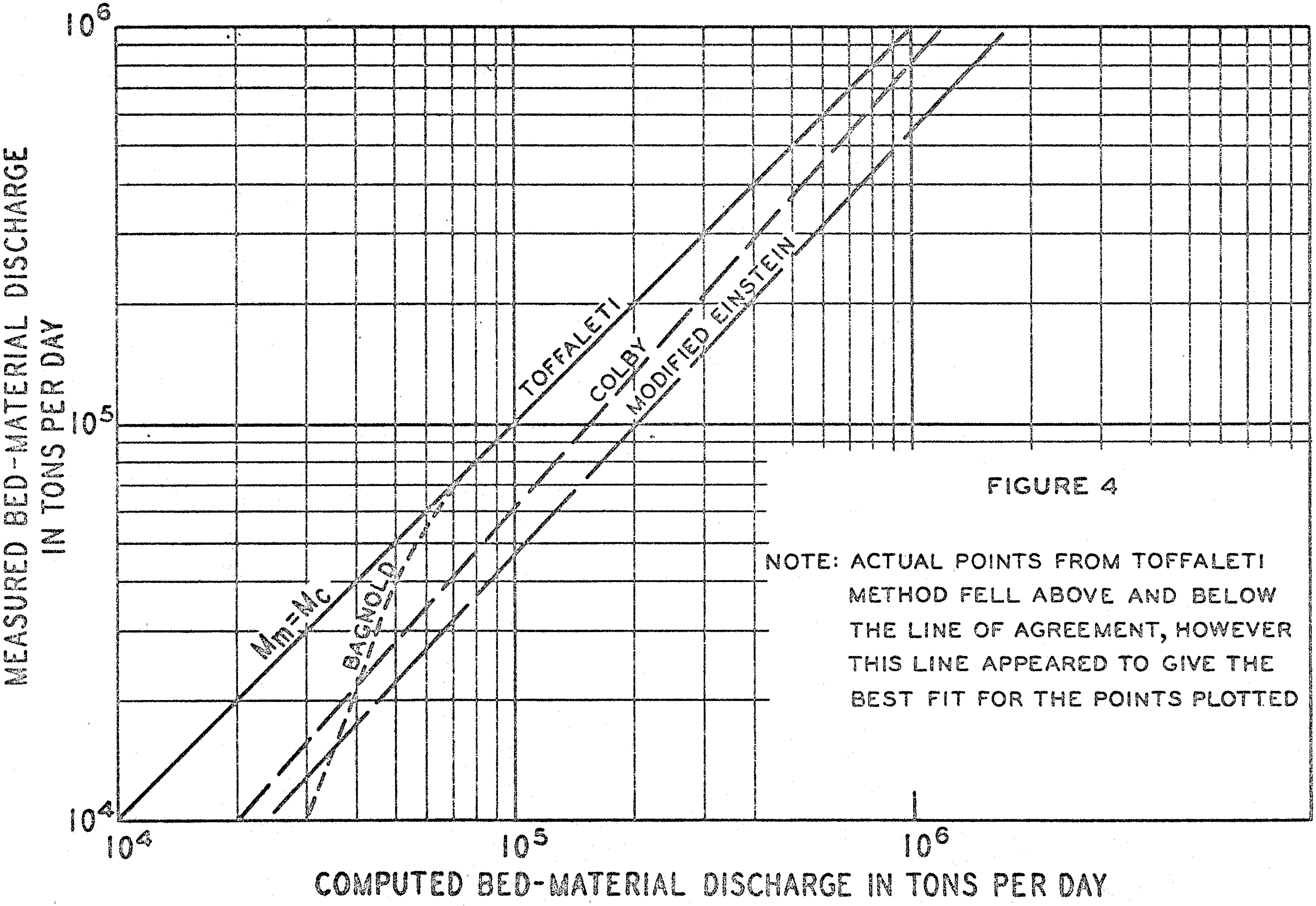


FIGURE 4

NOTE: ACTUAL POINTS FROM TOFFALETI METHOD FELL ABOVE AND BELOW THE LINE OF AGREEMENT, HOWEVER THIS LINE APPEARED TO GIVE THE BEST FIT FOR THE POINTS PLOTTED

TABLE 2

Cross-sectional Area Study of Range Sites, Dam No. 27

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Range No.	River Mile	Survey of 1951	Survey of 1952	Difference between (3) & (4)	Survey of 1958-61	Difference between (4) & (6)
	Mississippi River Silt Ranges					
1	191	36,576	39,840	+3,264	45,504	+5,664
2	192	42,560	47,200	+4,640	40,221	-6,979
3	193	26,944	46,208	+19,264	45,056	-1,152
4	194	32,224	41,536	+9,312	32,928	-8,608
5	195	31,712	31,264	-448	31,212	-52
	Missouri River Silt Ranges					
14	1	17,248	8,064	-9,184	18,752	+10,668
15	2	13,216	7,328	-5,888	18,067	+10,739

TABLE 3

Computed Bed-Material Discharges in Tons Per Day

Date	Measured Bed-Material *Discharge	Toffaleti	Modified Einstein	Colby	Bagnold
7/22/51	300,000	395,300	560,000	- -	- - -
7/16/51	250,000	353,300	470,000	370,000	530,000
5/23/51	94,000	129,300	190,000	160,000	210,000
7/30/51	46,000	145,900	120,000	- -	- - -
9/17/51	61,000	80,900	190,000	- -	120,000
8/19/52	5,600	7,500	21,000	15,000	29,000
12/29/52	2,000	1,800	2,900	5,000	16,000
1/13/53	750	1,400	940	2,800	20,000
7/16/53	25,000	15,200	63,000	46,000	30,000
8/10/53	15,000	11,100	34,000	30,000	23,000
3/22/54	4,400	4,000	9,100	9,800	14,000
7/26/54	3,700	6,000	11,000	9,400	22,000
10/18/54	44,000	129,500	93,000	73,000	53,000
2/24/55	110,000	75,500	240,000	170,000	47,000

*Bedload discharge not included.
Figures quoted are USGS.

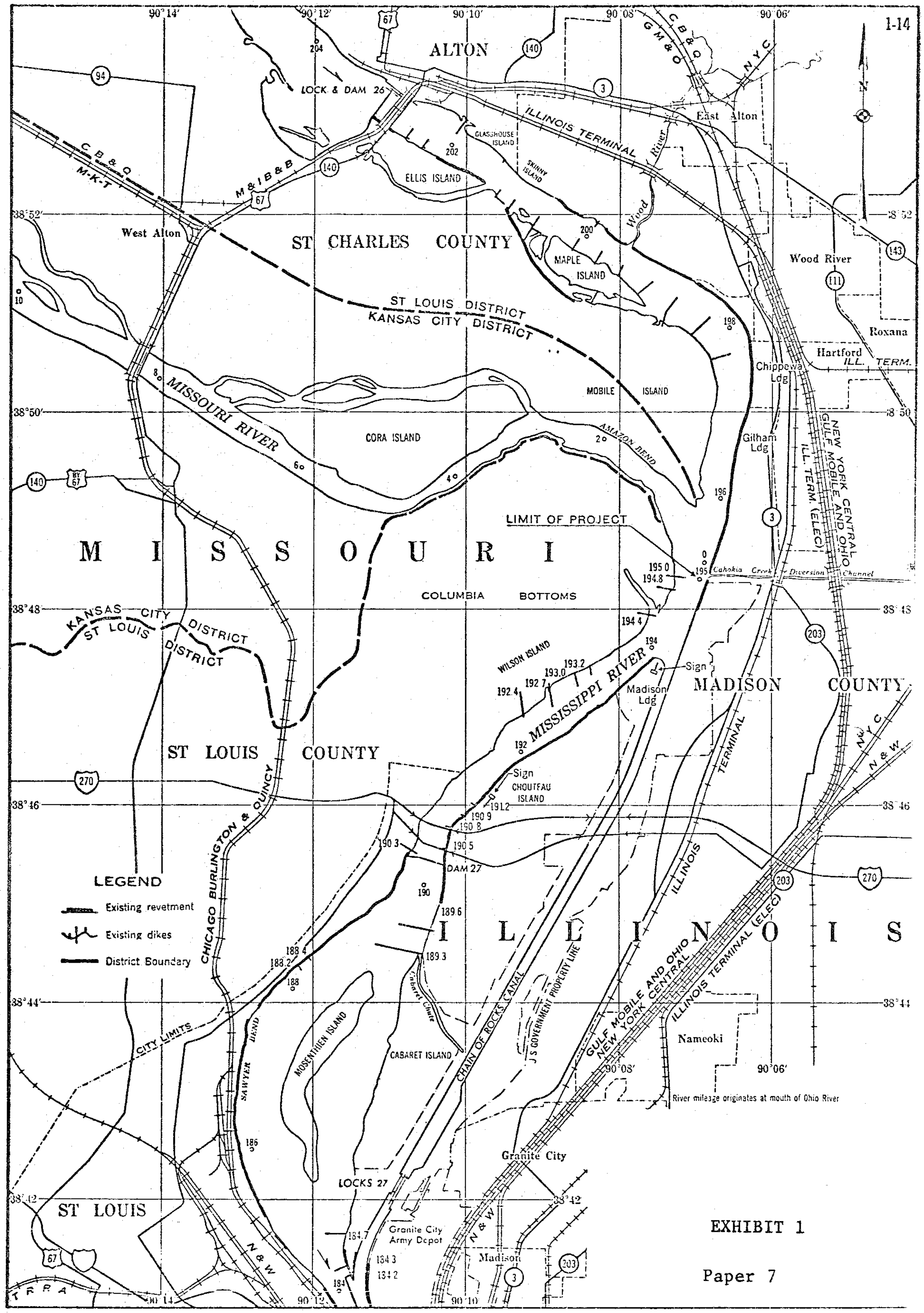


EXHIBIT 1

Paper 7

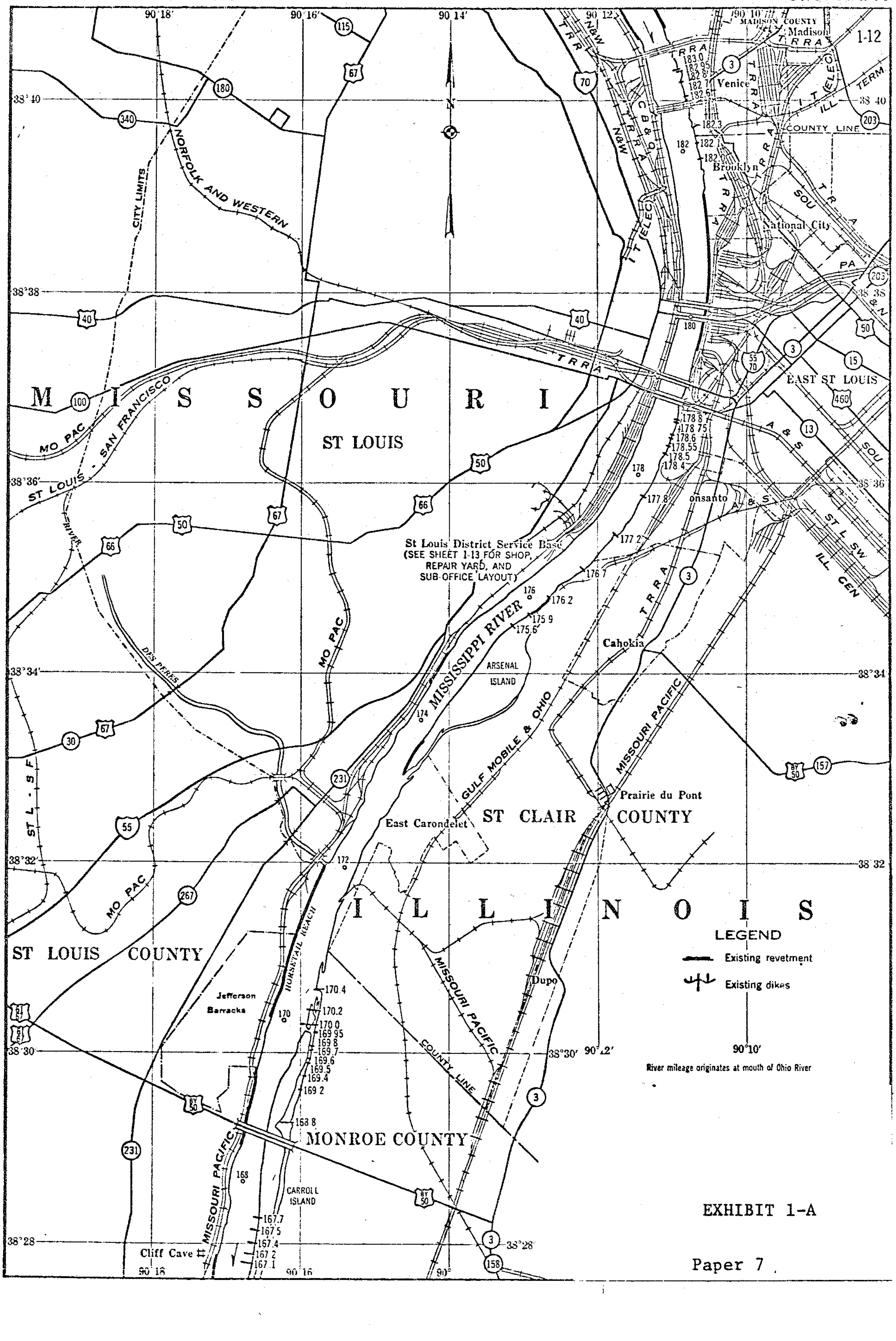


EXHIBIT 1-A

Paper 7

THE QUANTITY AND QUALITY OF SEDIMENTS DEPOSITED IN CLEVELAND HARBOR AT CLEVELAND OHIO

by

David L. Sveum[1]

CHAPTER I

INTRODUCTION

The United States Army Corps of Engineers is responsible for maintaining navigable depths in the harbors on the Great Lakes. Maintenance consists of dredging material from harbor areas and waterways to maintain the project depths. The Corps has been performing this work successfully for over one hundred years. In the past, excavated materials have been hauled to designated disposal areas in the Lakes and dumped.

In 1966, the Bureau of the Budget indicated the Corps should conduct a pilot investigation with the cooperation of the Federal Water Pollution Control Administration (FWPCA) and other agencies, to determine acceptable solutions to the dredgings-disposal problem.[1,4] Maintenance dredging of Great Lakes Harbors will have to continue even though dredging from many of the harbors contain considerable amounts of organic matter that could contribute to degradation of water quality of the Lakes.

For example, erosion of the Cuyahoga River valley and its tributaries bring large quantities of sediment to the main stream, which are carried into Cleveland harbor. Erosion of areas disturbed by construction also produces sediment. Considerable quantities of municipal wastes, flue dust and other forms of industrial waste are deposited in the harbor. All of the materials which are deposited in the navigation channel must be removed by maintenance dredging. The materials so removed are considered to be grossly polluted, and continuation of the historical practice of disposing thereof by pumping in deep waters of Lake Erie, is considered to be inimical to the ecology of the lake.

Purpose of Paper

It is the purpose of this paper to discuss some of the results of the pilot study and to focus attention on the problems of a specific harbor on the Great Lakes. Cleveland Harbor, situated along the lower reaches and at the mouth of the Cuyahoga River, and the Cuyahoga River basin have been selected as the topic for this paper because considerable data are available for this situation and the magnitude of the sedimentation problem in this harbor is one of the largest on the Lakes.

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Scope of Paper

Chapter II contains a discussion of the hydrologic characteristics of the Cuyahoga River basin.

Chapter III contains a discussion and the computation of the quantity of sediments deposited in Cleveland Harbor.

Chapter IV contains a discussion of the quality of sediments in Cleveland Harbor and the results of some FWPCA studies are also presented.

Chapter V presents a summary and conclusions.

CHAPTER II

HYDROLOGY OF THE CUYAHOGA RIVER BASIN

Basin Description

The Cuyahoga River basin lies in northeastern Ohio and drains an area of about 810 square miles. The basin is approximately "boomerang" shaped with a long eastern arm, as the result of drainage changes brought about during glaciation. The stream has its source about 33 miles northeast of Cleveland and flows in a southerly direction to a point near the village of Hiram Rapids, thence it flows southwesterly and westerly to its confluence with the Little Cuyahoga River at Akron, whence it flows generally north to Lake Erie at Cleveland. There is a breakwater protected outer harbor of about 1,300 acres at the mouth of the Cuyahoga River and the 5.8 miles of channel near the mouth have been improved for navigation. The basin contains portions of the cities of Akron and Cleveland and is one of the most heavily industrialized areas in the United States.

The watershed, except for the gently sloping area about three miles wide bordering Lake Erie, consists of rolling hills and contains some natural small lakes and ponds. In the upper reaches of the Cuyahoga River, above Cuyahoga Falls which is near the mid-point of the basin, the channel is shallow and is cut through glacial drift with a fall of about four feet per mile. At Cuyahoga Falls the river cuts through the Pennsylvania sandstone and drops a total of 220 feet in a distance of 1.5 miles. In the lower reaches the river flows in a preglacial valley, with a fall of about 5 feet per mile. A Cuyahoga River basin map, a project map of Cleveland navigation channel, and a project map of Cleveland outer harbor are shown on plates 1, 2 and 3 respectively.

Climate

The basin has a humid climate with precipitation distributed fairly uniform throughout the year. Mean monthly precipitation values at Cleveland vary from a minimum of 2.33 inches in February to a maximum of 3.49 inches in May. The average annual precipitation at Cleveland, based on a 96-year period, is 35.16 inches. The average annual snowfall recorded at four stations in and near the basin is 58.1 inches. Lake Erie moderates temperatures causing relatively cool summers and mild winters. The average annual temperature at Cleveland is 48.8 degrees Fahrenheit. July, the warmest month, has an average temperature of 71.0 degrees and January, the coldest month, has an average temperature of 27.6 degrees. The average growing season varies from about 163 days in the upland areas to 200 days on the lake plain.

Land Use

A large portion of the basin has been urbanized and the average population density is about 300 persons per square mile. Only eight percent of this population is engaged in agriculture. The major source of agricultural income from the watershed is from the sale of dairy cattle and dairy products. Poultry, truck and greenhouse enterprises are of significant importance. Approximately 41 percent of the farms have farm forests which produce stumpage for lumber and specialty products. Much of the farm land in the watershed is depleted in fertility and organic matter. All of the soils respond favorably to good crop rotations and soil treatment.

Soils

The upland soils in the basin have developed from glacial till. These soils have silt or clay loam textures with slow internal drainage. Along the flood plains of the streams, on glacial outwash areas, and areas that were occupied in prehistoric times by Lake Erie, the soils are partly of lacustrine and partly of alluvial origin. These soils have loam, sandy loam, or gravelly loam textures. There are small, scattered areas of poor drainage where peats and mucks have developed.

Runoff Characteristics

Runoff characteristics vary widely throughout the Cuyahoga River basin. A relatively distinct excarpment divides the basin between an upland plateau and the lake plain. Flows in the upper basin are modified to a great extent by existing reservoirs, relatively flat topography and some natural lakes. Discharges in the downstream basin resulting from rainfall over the upper Cuyahoga River basin may be reduced by regulation of the four water supply reservoirs in the upper basin. The combined drainage areas and storage capacities at these four reservoir sites are 265 square miles and 38,060 acre feet respectively. Flood peaks near the mouth resulting from rainfall over the entire basin occur principally as a result of runoff from the downstream portion of the basin. In recent years runoff from the upstream basin has contributed only 10 or 20 percent of the maximum discharges recorded in the downstream basin.

Streamflow

Streamflow in the Cuyahoga River basin follows a characteristic seasonal pattern. Fall and winter flows are generally low. There is a marked rise in discharge during March and April by runoff from the winter's melting snowpack and ice cover. Runoff is normally well sustained during April, May and June. During late summer the streamflow is quite low. Heavy rains may cause sharp rises during any of the spring, summer and fall months. The U.S. Geological Survey maintains a stream gaging station at Independence, Ohio, located about 14 miles upstream from the mouth of the Cuyahoga River. The drainage area upstream from this gage is 707 square miles. This station, which has 38 years of record, is the best available source of streamflow data for Cleveland harbor studies. Flows at this station have varied from a minimum of 14 cfs on 30 November 1930 to a maximum of 24,800 cfs on 22 January 1959. The average annual runoff from the basin upstream from the gage is about 14 inches.

CHAPTER III

QUANTITY OF SEDIMENT IN CLEVELAND HARBOR

Introduction

The volume of sediment accumulating in a harbor is influenced by the climate, topography, and land use within the basin. Erosion of agricultural lands generally supplies the greatest amount of sediment to streams. Erosion of stream banks, discharges from storm sewers, and discharges of industrial waste are additional sources of sediment. As stream-transported material enters the relatively quiet waters of navigation channels and harbors, it deposits and forms shoals. The bed load and heavier sediments are deposited near the head of navigation. Finer material is dispersed along the navigation channel and harbor bottom.

Sources of Sediment in Cleveland Harbor

In 1952 the Department of Agriculture made a study of the erosion and sediment damage which occurs in the Cuyahoga River watershed.[7] Erosion damage includes that which occurs as a result of sheet erosion, gully erosion, streambank erosion, and floodplain scour. Sediment damage includes that which occurs as a result of infertile overwash, impairment of drainage, sedimentation in ponds and reservoirs, damage to transportation facilities, sedimentation in drainage ditches, and damage to municipal water supply systems. The results of the Cuyahoga River watershed study by the Department of Agriculture shows that 28.1 percent of the total sediment reaching Cleveland Harbor comes from stream bank erosion; 15.5 percent is contributed by sheet erosion; 8.1 percent comes from flood plain scour; and 0.3 percent is from valley trenching and gully erosion.

The remaining 48 percent was estimated to be supplied by municipal and industrial wastes. This estimate was based in part on a study of flue dust deposits by the Corps of Engineers which indicated that about 32 percent of the deposits in Cleveland Harbor were from that source. The investigation by the Corps showed that blast furnace operations of three major steel companies located on the river banks adjacent to the navigation channel are the source of the flue dust. Since 1952 there have been a number of changes in local blast furnace operations. Although substantive data are not available, it is believed these changes have markedly reduced the flue dust contribution to dredging requirements. The remaining 16 percent of the municipal and industrial deposits were supplied from other waste products. The Department of Agriculture study showed that 6.5 percent of the total sediment load upstream from the harbor originated as industrial and domestic waste so a small portion of the total estimated waste load has been duplicated in the computations. A summary of the estimated sources of sediment in Cleveland Harbor is presented in Table 1.

TABLE 1

SOURCE OF SEDIMENT DEPOSITS IN CLEVELAND HARBOR

Source	Percent of Total	
Contributions from Cuyahoga basin sources upstream from Cleveland		
Stream bank erosion	28.1	
Sheet erosion	15.5	
Flood plain scour	8.1	
Valley trenching	0.3	
Sub-total		52.0
Contributions from Cleveland municipal and industrial sources		
Flue dust from blast furnace operations	32.0	
Waste products	16.0	
Sub-total		48.0
Total		100.0

Volumes of Dredged Material

Three Buffalo District hopper dredges begin the outer harbor maintenance at Cleveland in early spring, as soon as ice conditions permit. As warmer weather reduces ice on Lake Erie, the hopper dredges are dispatched to other harbors. The outer harbor maintenance is completed before the last one departs from Cleveland, generally some time in April.

Contracts with private firms for maintenance dredging of the inner harbor channels provide for starting the work in the fall of one year and completing it in the spring of the following year. Under the contracts the upper mile of the Cuyahoga River channel is dredged in the late fall to three feet below project depth. This is the area where most of the sediment load from upper river is deposited; the extra depth provides room for storage of most of this load over the winter, concentrating it for ease of dredging in the spring. The fall work requires about two months, the spring work about four. The contractor uses the following plant: a clamshell dredge, one tug, and four to six dump scows. The tug is used both for moving the dredge and dump scows in the harbor and hauling the loaded scows to established open lake areas in Lake Erie.

The volumes of dredged material that are transported from the harbor are recorded and are published in the annual report of the Chief of Engineers. A summary of data for the ten-year period from 1959 through 1968 is presented in Table 2.

TABLE 2

VOLUMES DREDGED FROM CLEVELAND HARBOR IN CUBIC YARDS

Year	Outer Harbor	Cuyahoga & Old Rivers (Spring)	Cuyahoga & Old Rivers (Fall)
1968	427,862	377,033	171,055
1967	510,327	525,000	200,000
1966	589,008	539,000	200,000
1965	560,174	495,000	200,000
1964	331,797	534,374	143,200
1963	393,420	508,000	230,000
1962	446,617	524,000	200,000
1961	630,306	557,000	186,000
1960	479,394	734,000	153,500
1959	762,411	615,134	199,996
Average	513,132	540,854	188,375

The data in Table 2 show that the average annual volume of sediment inflow to the harbor has been about 1,242,000 cubic yards. In addition, a small amount of permit dredging for dock owners at the harbor is accomplished by separate contract.

Grain Size

The Buffalo District, Corps of Engineers has sampled bottom materials at various locations in Cleveland Harbor. The materials generally fit the size range associated with silt and clay, although samples of bottom materials from just below the head of navigation on Cuyahoga River have shown the presence of significant amounts of coarser materials. Bottom materials elsewhere in the river navigation channel and in the outer harbor areas have been found to be fairly uniform and only slightly coarser than the suspended sediments at the upstream gaging station.

Suspended Sediment Studies

The U. S. Geological Survey takes samples of suspended sediment at their Cuyahoga River gaging station located at Independence. Samples are obtained periodically at various discharges and the suspended sediment load is computed. Attempts to measure bed load at this site have been unsuccessful. Annual discharges and suspended sediment loads recorded at the Independence site are presented in Table 3.

Based on their measurements for the period from October 1950 through September 1968, the average annual suspended sediment load at the gage site was 200,460 tons. Converting this tonnage to cubic yards (scow measure) using a dry-weight density of 50 pounds per cubic foot and a 15 percent bulking factor from in-place to scow measure, results in a corresponding quantity of about 342,000 cubic yards. Assuming that all of this material is deposited in Cleveland Harbor, it would account for about 28 percent of the total dredging required.

Annual Sediment Volume Deposited in Cleveland Harbor

Although the determination of the total amount of sedimentation in Cleveland Harbor is comparatively simple and dependable, the determination of the relative amount of solids derived from each of several major sources presents a more complex problem incapable of exact solution with available information. Some general estimates can be made based on the data presented in previous paragraphs. The total volume can be determined from records of quantities of dredged material. The volumes of flue dust and other waste products can be based on the relative percent of these materials in the samples. The volume of suspended load can be determined from the USGS

TABLE 3

ANNUAL DISCHARGE AND SUSPENDED SEDIMENT LOADS
FOR THE CUYAHOGA RIVER AT INDEPENDENCE

Water Year	Annual Discharge In cfs days	Annual Suspended Sediment Load (Tons)
1951	381,031	260,463
1952	393,198	270,258
1953	153,582	39,832
1954	204,503	150,827
1955	324,871	243,812
1956	408,366	321,614
1957	310,576	254,813
1958	279,384	137,065
1959	428,280	237,031
1960	377,609	232,773
1961	280,496	293,832
1962	190,419	139,510
1963	210,225	181,162
1964	216,800	245,639
1965	205,814	126,135
1966	225,619	156,868
1967	249,605	141,503
1968	264,773	175,146
Average Annual	283,620	200,460

measurements. The remaining volume, which is not accounted for in these computations, is assumed to occur as a result of suspended sediment from the ungaged area and from bed load. A summary of the estimated breakdown of sediments deposited in Cleveland Harbor is presented in Table 4.

TABLE 4

ESTIMATED BREAKDOWN OF ANNUAL SEDIMENT
VOLUME DEPOSITED IN CLEVELAND HARBOR

Source	Percent of Total	Volume in cubic yards
Flue Dust	32	398,000
Other Waste Products	16	199,000
Eighteen year average suspended load measured by the USGS	28	342,000
Estimated bed load and additional suspended load from the area downstream from the gaging station	24	303,000
Total	100	1,242,000

CHAPTER IV

QUALITY OF CLEVELAND HARBOR SEDIMENTS

Sources of Sediment Pollution

Sediments become polluted before, during, and after transport into harbors. In general, the sources of pollution can be categorized as municipal, industrial, and agricultural. Agricultural pollution derives from animal wastes and from the use of fertilizers and pesticides, whose residues are washed or leached into streams and other drainage channels. Some contamination of sediments and water must be ascribed to municipal wastewaters that are emptied into receiving waters as stormwater discharges, overflows of combined sewers, spillage from surcharged and broken sewers, and effluents from sewage treatment plants. Industrial waste-waters may be added along with municipal sewage or directly through industrial outfalls.

Pollution In Cleveland Harbor

The lower river and navigation channel throughout the Cleveland area is described by the FOPCA as being a virtual waste treatment lagoon. At times the river is choked with debris, oils, scum, and floating organic sludges.

Foul smelling gases can be seen rising from decomposing materials on the river's bottom. Viewed from the city's observation towers, the river appears to be chocolate-brown or rust-colored. During most of the year this section has no visible life, not even low forms such as leaches and sludge worms which usually thrive on wastes.

The inadequately treated wastes from the Cleveland Southerly Treatment Plant, and an undetermined number of storm water overflows and sewage bypasses discharge large quantities of oxygen-demanding wastes and bacterial contamination to the lower river. These domestic wastes are joined by the discharges from the major industrial complex in the Cleveland area. Steel and chemical companies discharge solids, nickel, fluorides, iron, oil, sulfates, ammonia, acids and other deleterious materials into the lower river.

The outer harbor receives the discharges from the Cuyahoga River and numerous storm water and combined sewer overflows. The water quality varies with meteorological conditions, especially the wind which frequently permits lake water to enter the harbor. Due to density differences, lake water frequently underruns or overruns the water of the Cuyahoga River and outer harbor.

Collection of Samples

Samples of bottom sediment for chemical and biological analyses were collected with Petersen, Ekman, Shipek, or Smith-McIntyre samplers. The samplers pick up material from the top several inches of the sediment and are believed to be representative of the material removed in maintenance dredging. Separate analysis of the top, middle, and bottom sections of core samples were generally inconclusive. Material dredged by clamshell was sampled after the dredgings had been placed in scows for transported to the disposal area. Representative grab samples were obtained from different locations in the scow and were composited into a single sample.

Sediment Quality Analysis

Some of the analyses of the bottom sediment constituents are based on standard methods, some are modified standard methods and some are entirely new approaches.[6] A laboratory manual which explains the methodology used in a chemical analysis of the sediments was presented as Appendix C 10 of the pilot study report.[1] The distribution of the constituents at sampling stations throughout the navigation channel by the FWPCA are illustrated on Plates 4 and 5.

A summary of the average concentrations of bottom sediment consituents in the river, outer harbor, and central Lake Erie is presented in Table 5. Analysis of midlake sediments made in 1963 are used in this comparison.

TABLE 5

COMPARISON OF AVERAGE CONCENTRATION OF
SEDIMENT CONSTITUENTS FOR VARIOUS AREAS
(mg/g dry weight)

Constituent	River	Outer Harbor	Central Lake Erie
Chlorine demand	30	12	--
COD	240	95	41
BOD_5	15	5	1
Volatile Solids	124	65	63
Oil and Greese	35	8	0.4
Phosphorus	4	1.5	0.7
Nitrogen	5	1.6	1.9
Iron	110	45	35
Silica	550	720	---

Chlorine Demand

Chlorine demand (15 minute) was determined on a dry weight basis for bottom sediments, but has not been determined for lake bottom samples.

Cuyahoga River sediments have a high chlorine demand probably due to high ferrous iron content. Test results were erratic as might be expected, but upstream from a point one mile above the river mouth the demand averaged more than 30 mg/g. Using an average sediment density, the 15 minute demand per cubic yard of in-place sediment would be approximately 36 pounds. In the lower one mile of the Cuyahoga River the chlorine demand decreased rapidly to the level of the outer harbor where the demand is only about one-half that found in the river sediments.

Chemical Oxygen Demand

The chemical oxygen demand (COD) of the river sediments is high. Proceeding upstream this demand climbs steeply in the lower one mile of the river from an average of 70 to 170 mg/g. Above one mile the average climbs gradually to about 270 mg/g near the head of the navigation channel. An average for the entire river at Cleveland would be about 240 mg/g which is equivalent to about 290 lbs/yd^3 of in-place sediment. The COD of the outer harbor sediments averaged about 40 percent of that in the river.

Biochemical Oxygen Demand

The 5-day biochemical oxygen demand (BOD_5) test on sediments is not considered a very good test as performed for this study. The test involved initial stirring and then quiescence for five days. Results varied widely in the river sediments and toxicity may have played some part in the scatter. In addition some of the oxygen demand measured here is chemical in nature. The extent is not determined since IDOD was not measured.

The BOD_5 of the river sediments, as measured, averaged about 15 mg/g or 18 pounds per cubic yard of in-place sediments. It increased sharply within the lower mile and then climbed gradually to the head of the channel. BOD_5 values for the outer harbor were much more uniform and averaged about 5 mg/g. The BOD_5 values of sediments were only about 6 percent of the corresponding COD values.

Volatile Solids

Volatile solids in the Cuyahoga River followed a pattern similar to COD with a rapid increase upstream in the lower mile from about 50 to 100 mg/g dry weight. Above one mile the increase was gradual to about 135 mg/g in the upper two miles of the navigation channel. The average for the river was about 125 mg/g or 150 pounds per cubic yard on in-place sediment. The average concentration in the outer harbor was slightly less than 50 percent of the concentration in the river.

Oil and Grease

Oils and greases are the constituents of the Cleveland harbor sediments which cause the most offensive appearance. They were measured for this investigation by hexane extraction.

In the Cuyahoga River navigation channel oil and grease content is high. In the lower mile of the river the concentration climbs sharply from 5 to 25 mg/g of dry weight. In the next mile it remains relatively constant and then climbs to about 45 mg/g. In the upper mile of the navigation channel the oil concentration falls to about 35 mg/g. An average for the river would be about 35 mg/g or 42 lbs per cubic yard of in-place sediment. The oil and grease content of the outer harbor sediment is only about one fourth that of the content in the river channel.

Phosphorus

The phosphorus content of river sediments is high, on the order of 15 times the average content of land sediments which are not artificially enriched. River sediment phosphorus concentrations are lowest at the river mouth, rising to a point 3.5 miles upstream, then declining farther upstream. River sediments averaged about 4 mg/g which is equivalent to 4.8 pounds per cubic yard of in-place sediment. This is equivalent to all the known phosphorus discharges to the Cuyahoga River. It is assumed that iron-heavy waters discharged primarily by steel plants cause the phosphorus to precipitate. The phosphorus level in the outer harbor sediments was fairly constant and averaged about 1.5 mg/g.

Nitrogen

Total nitrogen in the Cuyahoga River sediment was time-variable. The first samples in March 1967 showed much higher nitrogen content, especially ammonia, than later samples, probably because of slower breakdown of ammonia in winter, resulting in accummulation.

The average total nitrogen content for all sampling in the river was about 5 mg/g or about 6.0 pounds per cubic yard of in-place sediment. In the outer harbor the nitrogen concentration was more uniform and much lower, averaging 1.6 mg/g dry weight.

Total Iron

The iron content of the river sediments is high. Only the first few samples have been analyzed for iron, but those analyses showed an average concentration of about 110 mg/g or 132 pounds per cubic yard above one mile from the mouth. Near the mouth the concentration drops to about 30 mg/g while the outer harbor sedinents averaged about 45 mg/g.

Silica

The amount of silica in the sediment is an indication of the portion contributed from inorganic land runoff. It is in general inversely related to the volatile solids content. The river sediments average about 440 mg/g dry weight and the outer harbor 720 mg/g. Silica accounts for about 59 percent of the total solids in the river sediments and 70 percent of the harbor sediments. This indicates that more than one-half of the total sediment is derived from land sources and is consistent with 1952 studies made by the Department of Agriculture.

Sediment Quality Criteria

A current study was made by the FWPCA to relate measured chemical parameters to field observations and to determine which parameters are most significant and reliable; and to establish whether the objective determination of chemical parameters is more meaningful than the highly subjective assessment of sediment quality by field observations. Two hundred samples from Lake Michigan harbors were examined in this study. These samples were given overall ratings of lightly polluted, moderately polluted, or heavily polluted by field observers. The ratings were correlated with measured values of different parameters, and preliminary ranges were chosen for the measured parameter to cover the overall field ratings as closely as possible. The preliminary findings are presented in Table 6.

TABLE 6

CHEMICAL PARAMETERS AS A MEASURE
OF POLLUTION OF SEDIMENTS

Parameter	Degree of Pollution In mg/kg Light	Moderate	Heavy
Ammonia (N)	0-25	25-75	Over 75
COD	0-30,000	30,000-60,000	Over 60,000
Total Iron	0-8,000	8,000-12,000	Over 12,000
Lead	0-125	125-300	Over 300
Oil and Greese	0-1,000	1,000-2,000	Over 2,000
Total Phosphorus	0-125	125-300	Over 300
Sulfide	0-20	20-60	Over 60
Volatile Solids	1-3%	3-6.5%	Over 6.5%
Zinc	0-40	40-60	Over 60

Based on these criteria Cleveland Harbor sediments are heavily polluted.

Other Sediment Quality Studies

The U.S.G.S. made X-ray analyses of three Cuyahoga River sediment samples. The total air-dried fraction was analyzed by X-ray fluorescence for elements from atomic number 13 (aluminum) through atomic number 92 (uranium). The samples were also analyzed for mineral content by X-ray defraction.

The University of Wisconsin did a series of bioassays of the sediments. Their studies show a relationship exists between the chemical nature of the sediments and their toxic and algal-growth-promoting potential. The sediments were catagorized as falling into one of five groups. Sediments in catagory 5 were toxic and limited algal growth; sediments in catagory 1, the "cleanest" sediments, were non-toxic and stimulated algal growth. Ammonia, COD, volatile solids and phosphate content of the sediments progressively increased from catagory 1 through 5. These data suggest that visual observations should be supplemented by objective criteria in order to determine the degree of pollution of sediments.

CHAPTER V

SUMMARY AND CONCLUSIONS

Cleveland Harbor has a critical sedimentation problem. The sources of these sediments are erosion from the Cuyahoga River basin, flue dust from blast furnace operations, municipal wastes and other forms of industrial waste. The average annual volume of the sediment inflow to Cleveland Harbor is about 1,242,000 cubic yards. Based on preliminary FWPCA criteria, these sediments are heavily polluted.

The cheapest effective method of disposal, as an alternate to open lake disposal, is the use of diked containment areas near navigation projects. The use of diked disposal areas for all Great Lakes harbors where the sediments are rated as being polluted, as suggested in the Corps' report, is under consideration at the Washington level.

For Cleveland harbor the possibility of a settling basin in the lower Cuyahoga River channel has been discussed. The data presented in Chapter III show that a significant portion of the sediment inflow to the harbor originates in the Cuyahoga River basin upstream from the navigation channel. Disposal by loading the material from the settling basin into trucks for transportation to landfill sites appears to have merit for this location and is being given further consideration.

Treatment of dredged material could be an effective method of reducing possible harmful effects of in lake disposal. Feasibility studies were made for the Buffalo District by a consulting engineering firm to establish possible treatment processes and their capital and operating costs. These studies indicate that treatment of dredgings prior to open lake disposal is substantially more costly than disposal in diked areas.

All potential solutions to the dredging disposal problem are more costly than unconfined disposal in the open lake. Since it is unlikely that there will be complete control of pollutional inputs within the next 10 years, the only immediate solution to the problem of pollution of the Lakes by open-lake disposal is to dispose of the dredged material in confined areas. Congressional authorization, including decision with respect to local interest participation in the cost of providing diked disposal facilities, and funding, would be necessary before this is undertaken.

The pilot program studies included construction of pilot scale diked disposal areas at a few locations. The Corps report suggested further studies should be made to obtain and analyze data and tests not included in this report. Studies on the dispersion of sediments in the Lake as well as more data on the overall loadings to each lake were suggested. It was also suggested that experimentation with disposal in underwater excavated areas and harbor aerating and mixing may be productive.

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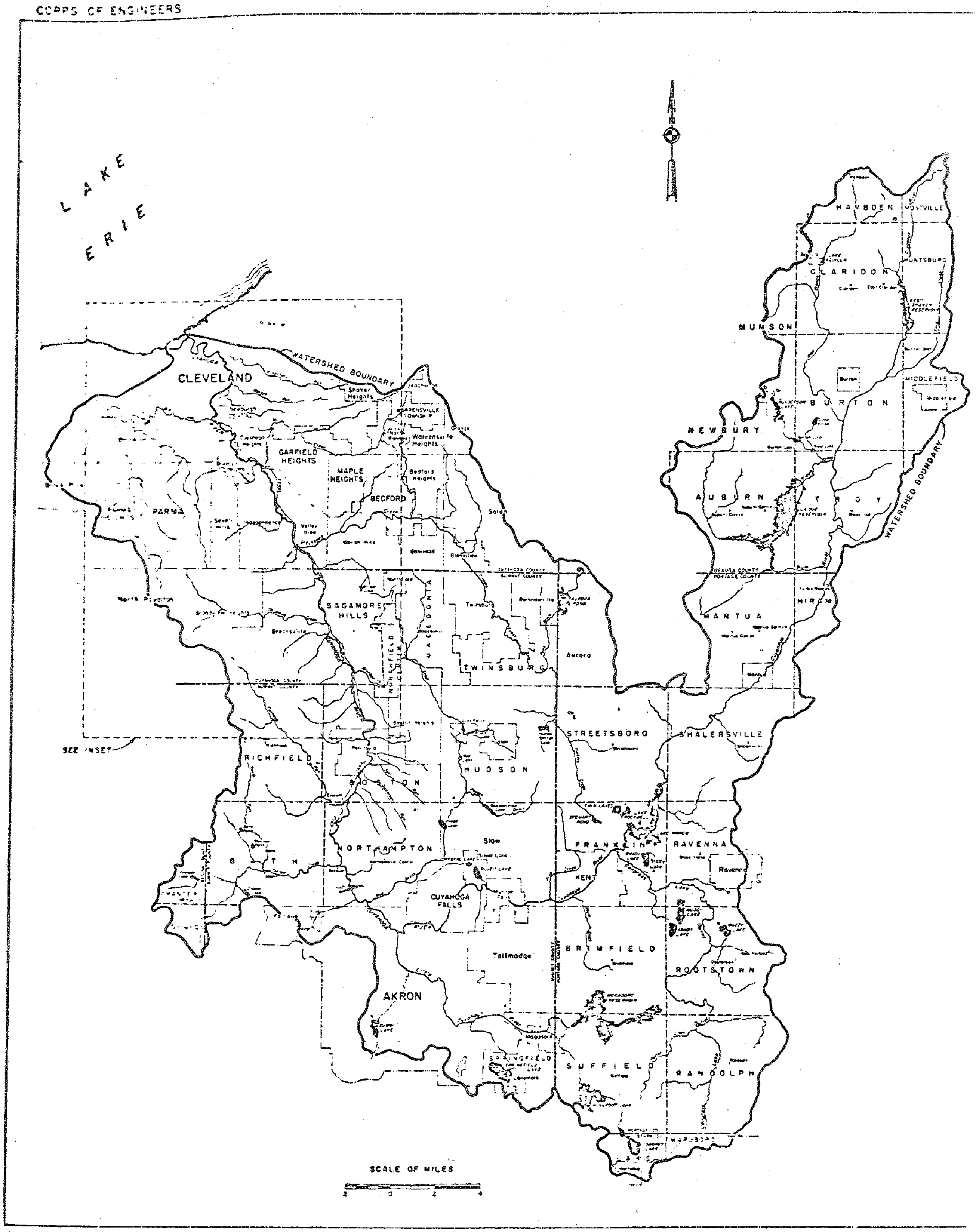
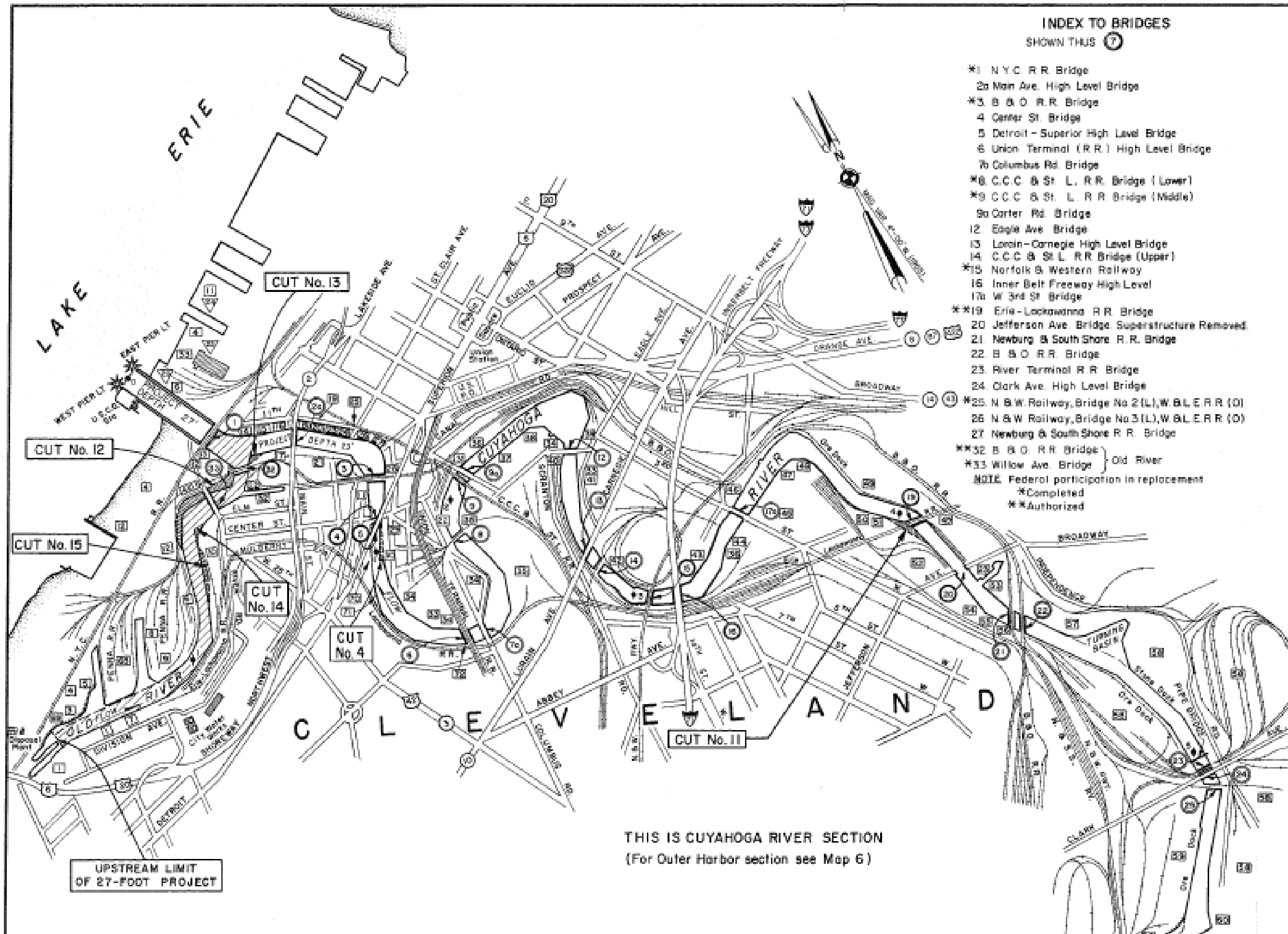


MAP OF CUYAHOGA RIVER BASIN

Plate 1



THIS IS CUYAHOGA RIVER SECTION
(For Outer Harbor section see Map 6)

INDEX TO BRIDGES

SHOWN THUS (7)

- *1 N.Y.C. R.R. Bridge
- 2a Main Ave. High Level Bridge
- *3. B. & O. R.R. Bridge
- 4 Center St. Bridge
- 5 Detroit - Superior High Level Bridge
- 6 Union Terminal (R.R.) High Level Bridge
- 7a Columbus Rd. Bridge
- *8. C.C.C. & St. L. R.R. Bridge (Lower)
- *9 C.C.C. & St. L. R.R. Bridge (Middle)
- 9a Carter Rd. Bridge
- 12. Eagle Ave. Bridge
- 13 Lorain-Carnegie High Level Bridge
- 14. C.C.C. & St. L. R.R. Bridge (Upper)
- *15 Norfolk & Western Railway
- 16 Inner Belt Freeway High Level
- 17a W. 3rd St. Bridge
- **19 Erie-Lackawanna R.R. Bridge
- 20 Jefferson Ave. Bridge Superstructure Removed.
- 21 Newburg & South Shore R.R. Bridge
- 22. B. & O. R.R. Bridge
- 23. River Terminal R.R. Bridge
- 24. Clark Ave. High Level Bridge
- *25. N. & W. Railway, Bridge No. 2 (L), W. & L.E. R.R. (O)
- 26 N. & W. Railway, Bridge No. 3 (L), W. & L.E. R.R. (O)
- 27 Newburg & South Shore R.R. Bridge
- **32. B. & O. R.R. Bridge } Old River
- *33. Willow Ave. Bridge } Old River

NOTE. Federal participation in replacement
*Completed
**Authorized

WATERFRONT OWNERSHIP

SHOWN THUS [4]

(O) Indicates Owner
(L) Indicates Lessee

1. Forest City Publishing Co. (O)
3. Esco Dredge & Fill Co. (L), N.Y.C.R.R. (O)
4. N.Y.C. R.R. (O)
5. Dunbar & Sullivan Dredging Co. (L) N.Y.C. R.R. (O)
6. Cleveland Stevedore Co. (L), Penna. R.R. (O)
7. Great Lakes Towing Co. (L), Forest City Publishing Co. (O)
8. Allied Oil Co. (L), Penna. R.R. (O)
9. Sand Products Corp. (L), Penna. R.R. (O)
10. Erie Dock Co. (L), Erie-Lackawanna R.R. (O)
11. City of Cleveland
12. Penna. R.R.
13. Tri-Mix Stone Corp. (L), Penna. R.R. (O)
14. Ryan Realty Co.
15. Joseph Waters (O)
16. Joseph Waters (O), Star Fish Co. (Operator)
17. National Terminals Corp. (L), B. & O. R.R. (O)
18. Stadium Parking Co. (O)
19. Housheer & Sons & Beacon Marine Suppy Co. Inc.
20. Housheer & Sons Co. (L), Eugene Kulber (O)
21. Cleveland Stevedore Co. (L), B. & O. R.R. (O)
22. Baltimore & Ohio R.R.
23. Great Lakes Towing Co.
24. Samsel Realty Co.
25. Interstate Mfg. Co. (L), Rudolph Lehotay (O)
26. Frank Morrison & Son Co.
27. West Superior River Front Co.
28. Berkshire Farm Inc. (L), Cleveland Trust Co. (O)
29. Joes Inc. (O), State Fisheries (Operator)
30. International Milling Co.
31. Mou-Sherwood Supply Co.
32. Huron Portland Cement Co.
33. C.C.C. & St. L. RR Co.
34. City of Cleveland
35. Upson Bolt & Nut Div. of Republic Steel Co.
36. Sherwin - Williams Co.
37. G. & W. Welding Co. (L), Mary S. Bradford (O)
38. Mary S. Bradford (O)
39. L. & M. Properties Inc.
40. Erie-Lackawanna R.R. Freight House
41. Brecking Supply Co. & Pionneer Materials (L), Arnold G. Breckling (O)
42. Ford Motor Co.
43. Standard Slag Co. (O)
44. Cleveland Gypsum Co. (L) Cleveland Builders Supply Co. (O)
45. Shell Petroleum Corp.
46. United Garage & Service Corp.
47. Texas Corp.
48. National Sugar Refining Co.
49. American Steel & Wire Div. of U.S. Steel Corp.
50. Clifton Concrete Supply Co. (L), W. & L.E. R.R. (O)
51. Socony Mobil Oil Co. (Operator) Socony Vacuum Co. (O)
52. Gulf Oil Corp. (O)
53. Standard Oil Co.
54. Cleveland Builders Supply
55. Marine Oil Terminal & Storage Co. (L), Byerlite Corp. (O)
56. Sinclair Refining Co.
57. Grosselli Chemicals Dept. of E. I. Du Pont De Nemours & Co.
58. Republic Steel Corp.
59. Jones & Laughlin Steel Corp.
60. Sun Oil Co. (O)
68. International Salt Co.
69. Cuyahoga Boat and Engine Co. (L), N.Y.C.R.R. (O)
70. Harry Rosenthal and Jack Malamud (O)
71. Erie - Lackawanna R.R. (O)
72. Fred Tod Jr. (O)

NOTES

Project depths: 27.0 feet in lower Cuyahoga River to junction with Old River and 23.0 feet in remainder of Cuyahoga River.
18.0 feet in Turning Basin at mile 4.8 on Cuyahoga River.
27.0 feet in Old River.

Project depths and soundings are referred to low water datum, elevation 568.6 feet above Mean Water Level at Father Point, Quebec (IGLD 1955) (International Great Lakes Datum 1955).

Miles above West Pier light at outer end of west pier shown thus 2 ♦

- [6] Indicates U.S. Routes
- (2) Indicates State Routes
- Indicates Interstate Route
- Indicates City of Cleveland dock numbering system

AREAS PARTIALLY DREDGED

Cuyahoga River partially dredged to 23.0 feet upstream of Bridge No. 1 to junction with Old River.

Old River partially dredged to 23.0 feet from junction with Cuyahoga River to opposite Sand Products Corp. dock, remainder dredged to 21.0 feet.

CLEVELAND HARBOR
OHIO

SCALE OF FEET

1000 0 1000 2000

CORPS OF ENGINEERS BUFFALO, N.Y.
30 JUNE 1967

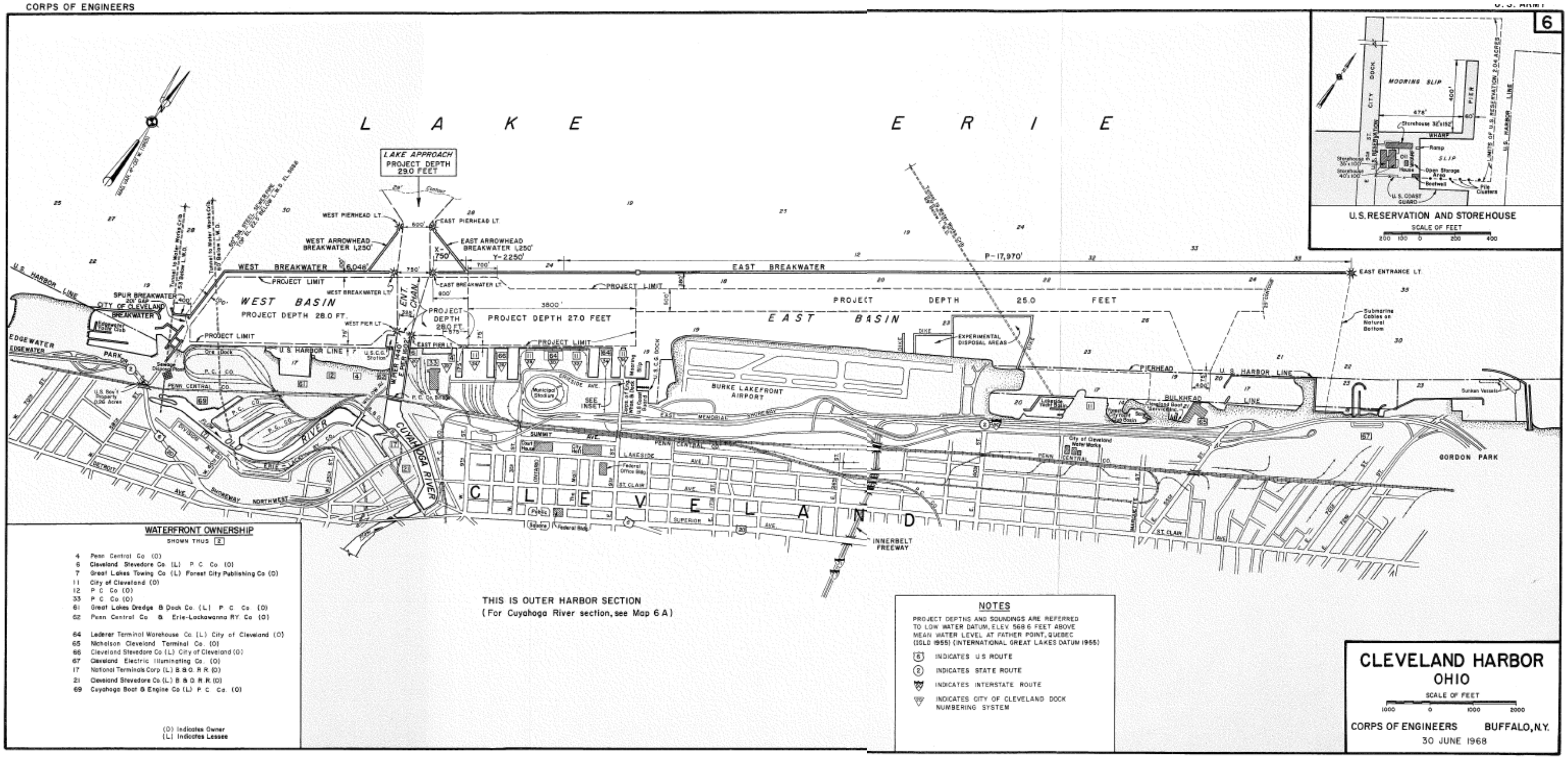

6
LAKE ERIE
LAKE APPROACH
PROJECT DEPTH
29.0 FEET
WEST PIERHEAD LT.
EAST PIERHEAD LT.
WEST ARROWHEAD
BREAKWATER 1,250'
EAST ARROWHEAD
BREAKWATER 1,250'
WEST BREAKWATER 6,048'
EAST BREAKWATER
P-17,970'
EAST ENTRANCE LT.
SPUR BREAKWATER
CITY OF CLEVELAND
BREAKWATER
EDGEWATER
PARK
WEST BASIN
PROJECT DEPTH 28.0 FT.
ENT. CHAN.
PROJECT DEPTH 28.0 FT.
PROJECT DEPTH 27.0 FEET
PROJECT LIMIT
EAST BASIN
PROJECT DEPTH 25.0 FEET
U.S. HARBOR LINE
EXPERIMENTAL DISPOSAL AREAS
BURKE LAKEFRONT AIRPORT
Municipal Stadium
SEE INSET
CUYAHOGA RIVER
PIERHEAD
BULKHEAD
GORDON PARK
INNERBELT FREEWAY
CLEVELAND
U.S. RESERVATION AND STOREHOUSE
SCALE OF FEET
WATERFRONT OWNERSHIP
SHOWN THUS 2
4 Penn Central Co (O)
6 Cleveland Stevedore Co. (L) P. C. Co. (O)
7 Great Lakes Towing Co (L) Forest City Publishing Co (O)
11 City of Cleveland (O)
12 P. C. Co (O)
33 P. C. Co. (O)
61 Great Lakes Dredge & Dock Co. (L) P. C. Co. (O)
62 Penn Central Co & Erie-Lackawanna RY. Co (O)
64 Lederer Terminal Warehouse Co. (L) City of Cleveland (O)
65 Nicholson Cleveland Terminal Co. (O)
66 Cleveland Stevedore Co (L) City of Cleveland (O)
67 Cleveland Electric Illuminating Co. (O)
17 National Terminals Corp (L) B. & O. R. R. (O)
21 Cleveland Stevedore Co. (L) B. & O. R. R. (O)
69 Cuyahoga Boat & Engine Co (L) P. C. Co. (O)
(O) Indicates Owner
(L) Indicates Lessee
THIS IS OUTER HARBOR SECTION
(For Cuyahoga River section, see Map 6 A)
NOTES
PROJECT DEPTHS AND SOUNDINGS ARE REFERRED TO LOW WATER DATUM, ELEV. 568.6 FEET ABOVE MEAN WATER LEVEL AT FATHER POINT, QUEBEC (IGLD 1955) (INTERNATIONAL GREAT LAKES DATUM 1955)
6 INDICATES U.S. ROUTE
2 INDICATES STATE ROUTE
INDICATES INTERSTATE ROUTE
INDICATES CITY OF CLEVELAND DOCK NUMBERING SYSTEM
CLEVELAND HARBOR
OHIO
SCALE OF FEET
1000 0 1000 2000
CORPS OF ENGINEERS BUFFALO, N.Y.
30 JUNE 1968

PLATE 3

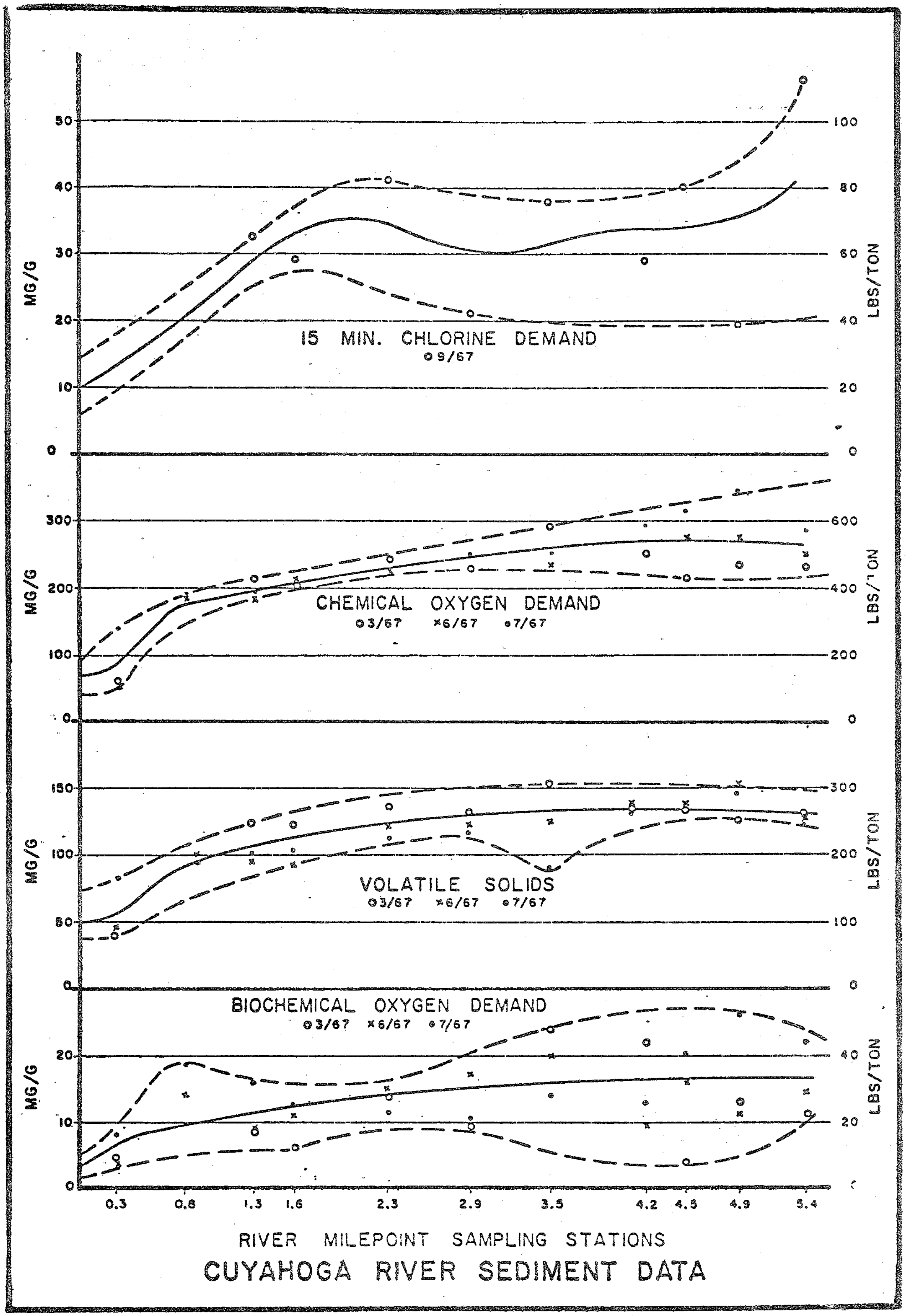
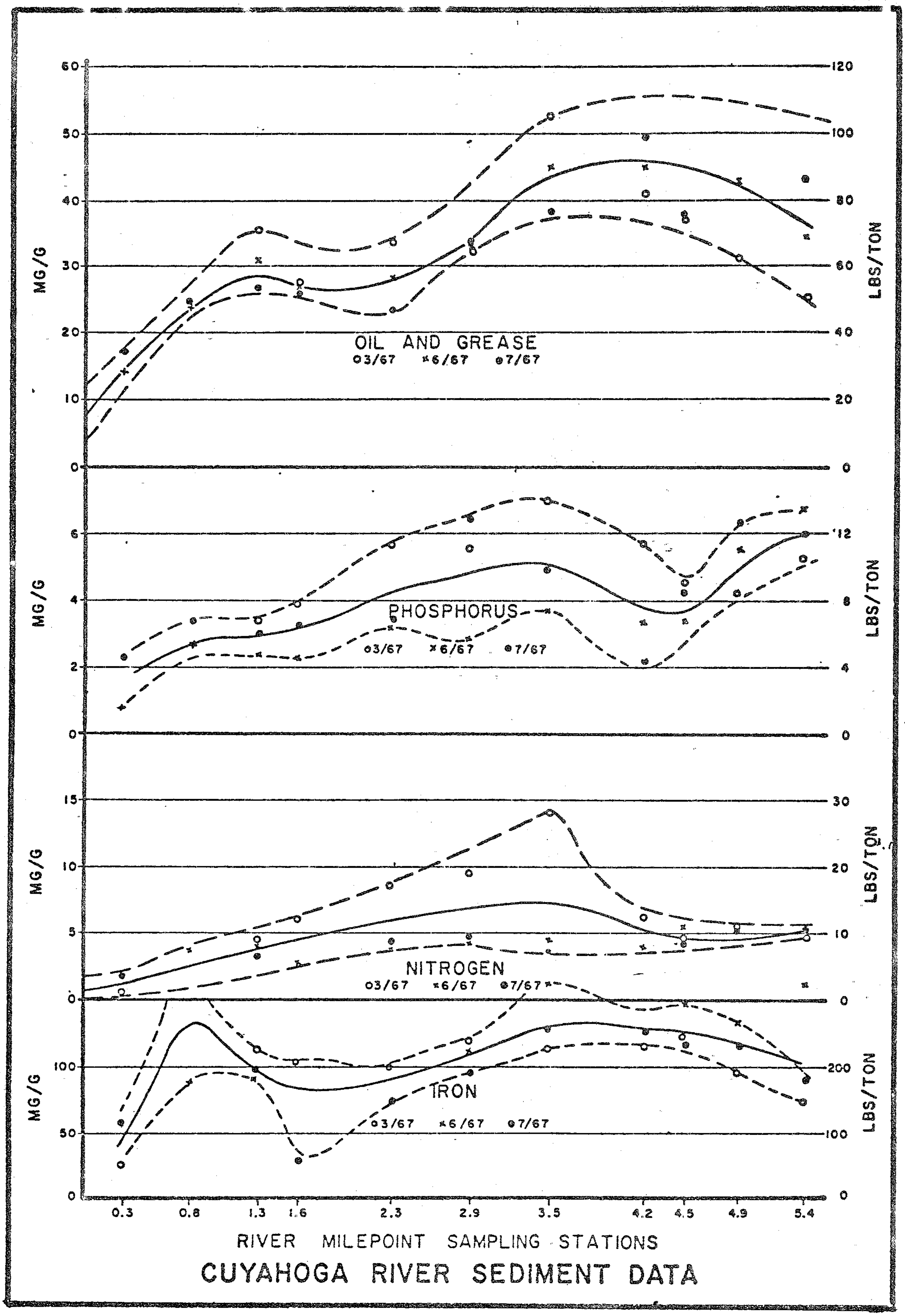

15 MIN. CHLORINE DEMAND
9/67
CHEMICAL OXYGEN DEMAND
3/67 6/67 7/67
VOLATILE SOLIDS
3/67 6/67 7/67
BIOCHEMICAL OXYGEN DEMAND
3/67 6/67 7/67
MG/G
LBS/TON
0
10
20
30
40
50
100
200
300
150
60
80
400
600
0.3
0.8
1.3
1.6
2.3
2.9
3.5
4.2
4.5
4.9
5.4
RIVER MILEPOINT SAMPLING STATIONS
CUYAHOGA RIVER SEDIMENT DATA

Plate 4



OIL AND GREASE
○3/67 ×6/67 ●7/67
PHOSPHORUS
○3/67 ×6/67 ●7/67
NITROGEN
○3/67 ×6/67 ●7/67
IRON
○3/67 ×6/67 ●7/67
MG/G
LBS/TON
0.3 0.8 1.3 1.6 2.3 2.9 3.5 4.2 4.5 4.9 5.4
RIVER MILEPOINT SAMPLING STATIONS
CUYAHOGA RIVER SEDIMENT DATA

Plate 5

THE ROLE SEDIMENTS PLAY IN DETERMINING
THE QUALITY OF WATER

by

Robert H. Livesey[1]

INTRODUCTION

Our society has become increasingly aware of the need for better control of its environment. Through legislative action, it is demanding increased efforts from all professions to develop rational methods of pollution abatement and control. This stimulus provokes a multitude of presently unanswerable questions and clearly demonstrates the need for greater basic knowledge of the many factors which determine our environment. Sedimentation is one of these factors. Sediments rank not only as a major cause of water pollution, but also play a predominate role in determining the quality of water. As a catalytic, transporting, or storage agent they also contribute to the seriousness of other forms of pollution. Yet, important interrelationships between many physical, chemical, biological, or other environmental aspects are either vague or unkown. It is apparent that an immediate need exists for the establishment of practical guidelines or priorities for anticipated study efforts; but, first, the sediment related problem areas must be identified.

The purpose of this paper is to identify such sediment related problem areas by citing and discussing a broad range of specific examples. It is not intended that these comments be focused only on engineering applications but, rather, that they be oriented toward all disciplines associated with water resource planning. It is hoped that, regardless of whether the problem is faced by the conservationist or biologist, the economist or planner, the lawyer or politician, or the chemist or engineer, these comments will present a clearer insight in to how sedimentation influences our environment.

IDENTIFICATION OF PROBLEM AREAS

It is intended that the real importance of this report focus on the identification of the numerous ways in which sedimentation processes influence the quality of water. This is difficult because the definition of a problem, its magnitude, and its complexity are all relative to the discipline identifying it. For this reason, an attempt has been made to distinguish several broad categories that represent common areas of interest between disciplines. Specific sediment

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related problems are then identified and associated with certain categories. Some examples are so diverse that they can be logically associated with several categories. For instance, problems related to eutrophication would also involve many other aspects and standardization of techniques could apply in various degrees to all categories.

Undoubtedly some problem aspects will be missing or perhaps only vaguely identified. This should be considered a challenge to others to add to the list. The author does not pretend to be qualified to establish a comprehensive listing for all disciplines; but it is thought that recognition of problem areas on an inter-disciplinary level is necessary to place some perspective on the importance of sedimentation in the over-all water quality picture. The following comments, perhaps, reflect some measure of the current state of the art of sedimentation as viewed by the engineer. Further amplification by other disciplines is essential and encouraged if a coordinated approach is to be taken toward the solution of problem areas.

The order in which the examples are identified does not necessarily indicate the level of relative priority. Although unintentional, some bias probably exists in certain comments because of their familiar nature. An effort was made to present only a concise summation that would identify each example without a voluminous background of information. In some instances a question was considered the best means of doing this.

Assimilation

The waste assimilation capability of both suspended and streambed sediments is paramount in this area. Exchange processes play the key role assisted by hydraulic conditions. The permanency of assimilation effects onto or into the sediment particle needs better definition. Some sediments are capable of assimilating waste without major changes in water quality or transport characteristics. The combination of sunlight with either inert or organic sediments is relevant.

An equilibrium balance apparently exists in the ion exchange rate between solutes and sediments. Various types of sediment seem to react in a different manner depending upon the molecular attraction of the dissolved solids content of the water. It is probable that a large "ionic load" might be transported in an absorbed state on sediments. The degree and type of additional downstream assimilation could well depend upon the mineral contribution of tributary streams.

Biological Environment

Probably the most important influence of sedimentation on the biological environment relates to the ability of the aquatic community to function adequately and vigorously. The influence covers an extremely broad range from bacterial processes to aquatic plants. Turbidity and deposition are key factors. Examples include respiratory deficiencies on embryos and larvae, abrasive action on delicate membranes, ingestion of toxic accumulations, burial of organisms, plugging of gravel spawning beds, the oxygen demand of organic residue, and the microbiology and compaction of sediments.

Excessive aquatic growth in lakes and shallow streams is posing a difficult and costly problem. It is associated with nutrient supply and sunlight penetration. The use of suspended sediments to cause turbid conditions is a possible means of control.

Deposition

Basically, sediment deposition is associated with predicting the depletion rates of reservoirs and lakes but it also includes the exchange processes that occur at the water-bed interface. The decomposition of organic matter, plus the release of nutrients from bottom sediments, stimulates biological growth which in turn causes severe oxygen depletion. Thermal stratification can serve both as a deterrent and a catalyst.

Another facet involves controlling and concentrating the deposition of undesirable sediments at designated locations within a body of water. Improvement of dredging spoil techniques is of immediate concern. Deposition control in large lakes or reservoirs by thermal acceleration needs investigation.

Eutrophication

Sedimentation plays a predominate role in the eutrophication processes of lakes and reservoirs, but answers are needed to define what types of sediment are influential. Nutrients transported by inflowing sediments are concentrated in bottom deposits and released to the overlying water. Accelerated biological activity by both plants and animals within the photosynthetic region produces an over production which in turn die off and accumulate with the bottom sediments. Decomposition releases the nutrients to start the cycle again, but the rate is influenced by the quantity and quality of additional sediment inflow. Deposition processes may either accelerate or depress the nutrient exchange level. Turbidity, or the lack of it, will influence the depth of the photosynthetic region.

Pollution and Toxicity

This classification overlaps assimilation to a degree, but it pertains mostly to organic, chemical, and radioactive pollutants. Since sediments provide large surface areas for chemical action to occur they may contribute significantly to the rapid degradation or detoxification of pollutants.

Pollutants attached to sediment particles are not dispersed or transported as rapidly as dissolved pollutants. Large concentrations may biuld up in the stream bed or reservoir deposits awaiting a significant change in water chemistry to release the concentration into solution. Similarily, such concentrations might be effectively removed from the water environment by burial in delta deposits. A prime example is radioactive pollutants.

Perhaps one of the more beneficial effects of sedimentation on water quality is the removal of pesticides from solution by clay particles. Apparently the chemical exchange capacity of these sediments accommodates the requirements of these highly toxic pollutants. A comparable, but less defined aspect might be the control of acid drainage from mine wastes.

Identification of the trace minerals or metals constituting the sediment pollutant is a major area. Information on the movement of such contaminants and their life cycle is needed to locate source areas, predict yields and suggest methods of abatement. The hydrologic and erosion processes of sediment transport play a key role.

Sediment Yields

Major advancements have been made in soil conservation techniques and practices, but many questions remain. Primarily they relate to the source, rate, and management of surface soil erosion. Examples are vegetative influence on raindrop impact and erosion velocites; identification of soil stability characteristics and their relation to the mechanics of erosion; delineation of uniform source areas; sediment reduction techniques applicable to urban development, highway construction and harvested forest lands; practical selection of recreational sites; and many others.

Standardization of Techniques

This category applies equally to all disciplines. There is a marked inconsistency in both the methods and terminology employed in sediment related work. The definition of standard guide lines or methods is urgently needed in order to validate comparative conclusions arrived at by other disciplines. Current laboratory techniques are probably acceptable in many instances, but measurement methods are both variable and incompatible. For example, turbidity is currently measured by the laboratory analysis of sediment concentration or the visual sighting of a Secchi disc or Jackson candle unit. To some degree all are vague.

Improvement has been made on monitoring instruments to measure many water quality parameters associated with sedimentation processes. Automation of sampling procedures and laboratory analysis techniques deserves greater attention.

Sedimentation is not entirely detrimental so there must be standards to differentiate the good from the bad and the degree. Recommendations and guide lines are needed to assist authorities in the proper definition of sediment pollution. This can be implemented through the present system of "state standards".

<u>Transport</u>

Basically, this category concerns the movement of sediment, both organic and inorganic, in a stream by turbulent flow. Associated with it would be the influence of dissolved solids concentrations, thermal changes, bed forms, and hydraulic characteristics.

The thermal aspects associated with nuclear power plants is of current interest. An immediate appraisal is needed relative to the effect of long term thermal changes on the sediment transport capabilities of a stream including the geo-chemical behavior of polluted sediments. Bed form roughness is also associated with changes in water temperature. Such roughness determines the rate or time of storage of contaminated sediments in the stream bed.

The effect of suspended sediment on the diffusion of turbulent energy is a broad area of concern. Bed forms again play a key role in the magnitude and distribution of energy throughout a spectrum of frequencies. Effective waste assimilation or dispersion of pollutant inflow requires such energy transfer.

<u>Turbidity</u>

Turbidity has been recognized in several other categories, but it is also related to flocculation processes, taste and odor problems in municipal water supplies and the energy budget of lakes and streams.

Not only is there a need for standard definition but also instrumentation that will consolidate the concept of turbidity into a more meaningful entity.

The energy budget of bodies of water depend upon the scattering and absorption of various wave lengths of light. Turbidity levels limit or restrict this light penetration. Similarly, the flocculation characteristics of streamborne clay particles can limit algae growth in shallow streams. Without this benefit, the control of taste and odor in water treatment processes can be expensive.

Future water needs anticipate a significant source from desalinization. Flocculation techniques will enhance sediment removal prior to treatment processes. Irrigation practices have always recognized sediment removal as a major problem, but now there is a need for classifying turbid irrigation waste water before returning the flow into a stream.

NEEDS AND CAPABILITIES

It should be apparent from the above comments that the role sediments play in determining the quality of water is significant. But how do we go about resolving such a diverse range of problems? What are our needs and how do they compare with our capabilities?

Organizations concerned with water resource development are expected to have a competent staff that is cognizant of planning requirements on both a broad and a specific base. Where competence might be lacking, consultation is sought from specialists. It is here that the Sedimentation Engineer, along with the Limnologist, Chemist, Biologist, Conservationist and others, is called upon for his expertise. But it is also at this junction that the lines of communication become warped or entangled. The recognition of one or another's needs, efforts, or accomplishments is slow in filtering throughout such a communication net. It is not a matter of lack of interest or initiative on the part of these specialists. Each is eager and willing to communicate in order to resolve his particular problem or assist in someone elses, but many times it requires major detective work to ferret out who is accomplishing what. This is particularly true between organizations, agencies, or societies but it also occurs too frequently within such groups.

Obviously each discipline involved in water resource planning has specific needs oriented toward its own particular problems. Through individual efforts, progress will slowly advance with sporadic thrusts of advancement here and there as concerted attempts achieve a breakthrough. It is probable that these efforts will not diminish even by expressed urgencies originating from outside the individual discipline; it is not necessarily intended that they should. But some recognition must be given to the over-all water quality picture in order to place such needs in perspective for determining environmental planning priorities.

The manner in which this can best be accomplished would be on an inter-disciplinary basis. The organization level at which such an inter-disciplinary communication net should focus would probably depend upon the project scope. In some instances it might converge at the project planning level, in another at the design stage, or even at a rehabilitation plane; this is somewhat immaterial. The important point is that the capabilities of relevant disciplines be realized and lines of communication established across the working level. Task or work groups are common examples of such effort. They are usually effective in accomplishing an assignment, but the formality of organization and the functioning level is often unwarranted for the type of problems previously identified. A more direct consultation approach seems prudent and possible once the proper communication net is established.

CONCLUSION

Individual water resource disciplines are well aware of their technical capabilities, progress achievements, and potential needs. The sedimentation profession is a typical example. But the sediment oriented problems associated with water quality pollution and control are inter-disciplinary in nature. Many are definable, others are vague and some unrecognized at present. Obviously the coordinated attention of several disciplines will be required for solution. Such effort demands recognition of one another's capabilities through lines of communication that still need to be established. The Sediment Engineer must recognize that his professional existence is not limited to endeavors of the past. He must expand his vision to recognize not only his own future needs, but also the need of his expertise by others.

THE INTERRELATIONSHIP BETWEEN WATER TEMPERATURE, BED CONFIGURATION, AND SEDIMENT CHARACTERISTICS IN THE MISSOURI RIVER

by

Warren J. Mellema[1]

INTRODUCTION

Much research has been directed toward the development of prediction equations for the determination of sediment discharge in sand bed streams. Most of the developments to date involve examination of results of detailed flume or laboratory investigations to categorize various factors which influence the nature of its suspended load. This is accomplished by holding basic functions reasonably steady and uniform and noting the changes in the remaining variables. Often it is necessary to group several of the functions together into dimensionless parameters and investigate these combinations. This has the effect of decreasing the number of parameters requiring observation, but does not necessarily separate the effects of the individual parameters.

The above approach has definitely led to a better understanding of the basic principles involved in sediment transport and, at the same time, has revealed to the investigator the extreme complexity of the sediment and streamflow interrelationships. The large number of variables to be considered compounds the problem of developing reliable prediction equations. As a result of this, the investigator is forced to combine, or sometimes overlook, certain factors known to have an influence on the problem because of the inability to accurately measure or define the property in the field.

IMPORTANT VARIABLES

Listing variables that may influence the sediment load or discharge at a given location is much easier than separating their effects. At a given section, the sediment discharge may be considered to be a function of the size, density, shape, and cohesiveness of the bed and banks of the stream; the geology, hydrology, meteorology, topography, soils, and vegetal cover of the drainage basin; and the width, velocity, energy gradient, temperature, and turbulence of the flowing water.(1) Some of these parameters are not only nonuniform with respect to time, but also vary laterally in the cross section and vertically throughout the depth of the channel. This necessitates the use of average values and judicious assumptions in order to reduce the problem to one of manageable proportions.

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The ultimate usefulness of a laboratory investigation should be measured by the investigator's ability to reproduce or simulate actual prototype conditions. If, for example, the measurements and conclusions based on laboratory studies are too complex or require totally unnatural physical conditions, their usefulness may be purely academic; but, if these same conclusions can be verified in a natural stream, we have truly gained an insight into the total problem.

MISSOURI RIVER STUDY

Major river systems controlled by systems of upstream reservoirs offer unique opportunities to not only verify laboratory results, but possibly expand on them with actual field data. Measurements obtained on streams where the discharge is held reasonably constant over a sufficient length of time tend to dampen out, or sometimes even eliminate, many of the normal fluctuations found in most natural streams. Selected reaches of the Missouri River fit into this category, and the Omaha District of the Corps of Engineers has been conducting detailed field investigations on one of these reaches. MRD Sediment Series Report 13B[2] entitled, "Missouri River Channel Regime Studies" presents the results of the first three years of this study, and is used as a basis for the remainder of this discussion. Plate numbers appearing in this paper refer to the above reference.

The study was originally initiated to investigate factors that might be influencing periodic shifts in the stage discharge relationship of the Missouri River. This phenomena has been noted in numerous streams and has been documented many times in literature.[3, 4, 5] Of particular interest, however, are changes that occur in some of the associated hydraulic and sediment characteristics of the channel during these periods. Because of the large number of closely related parameters which dictate the ultimate regime of an alluvial stream, the net result of a stage shift may be tied to several interrelated variables. Conceivably, a shift could be triggered by any one of several factors and this triggering mechanism may not necessarily be constant for diverse streams.

One of the most significant observations revealed by the investigation is illustrated on Plate 16[2]. A downward shift in the stage discharge relationship at the Omaha gaging station of about 1-1/2 to 2 feet occurs in the fall months of the year with little or no variation in the discharge of the river. Closely associated with this shift is a gradual lowering of the river water temperature. This observation has been documented at the Omaha station each year from 1966 through 1969, and may very well be the triggering mechanism that sets off a chain reaction of further changes in the total regime of the river.

Some of the more pronounced changes that have been documented have been associated with the stream's sediment transport characteristics. The temperature of the water affects a stream's ability to transport sediment primarily by its large influence on the viscosity of the fluid. The most immediate effect of a change in viscosity is the marked influence on the fall velocity of sand particles in suspension. A decrease in the water temperature increases the fluid viscosity and decreases the fall velocity of suspended particles. This causes large size particles at high water temperatures to act like small size particles at the cold water temperatures. This change is more significant for the larger size particles than for the smaller size particles, since the stream already has the ability to transport all of the available small size material. If a sufficient number of particles in certain critical size ranges are available in the stream bed, a dramatic change in the total sediment load of the stream can occur. This increase in the amount of material in suspension has an even further effect on the fluid properties. Vanoni and Nomicos[6] have reported that the mere presence of fine sediment in the flow tends to further decrease the fall velocity of the coarser suspended particles. A major change in the water temperature can therefore have a very significant influence on a stream's ability to transport material in suspension.

The principles stated above appear to be verified by the Missouri River investigations. Detailed suspended sediment measurements at an established range in the study reach not only have shown a substantial increase in the total amount of material in suspension as the water temperature drops, but also indicate that changes occur in the size distribution of the material in suspension. Plates 12 and 13[2] show that the total concentration of all sand size particles in suspension (material coarser than 0.053 mm) increased from about 350 ppm in August to nearly 700 ppm in November as the water temperature decreased from 80°F. to 40°F. This increase in concentration occurred in a period of nearly constant flow, therefore resulting in a 100% increase in the total load passing the measuring section.

Plates 50 through 54[2] illustrate the effects of changes in water temperature on the size distribution of the suspended material. The graph entitled "Distribution of Grain Sizes for Bed Samples and Suspended Load" shows the distribution of the load in each size fraction, and relates this with the percentage of this same size material found in the bed of the stream. Analysis of these charts indicates that in August and September, (Plates 50 and 51), a larger portion of the suspended material was made up of the finer grain size fractions, while in the later period, (November), the proportion of fines declines with a corresponding increase in the medium and coarser grain size fractions. The most significant change appears to be in the 0.105 to 0.149 mm size range. In August and September about 25 to 30 percent of the total load was comprised of particles in this size range, while in late November, nearly 50% of the entire sediment load passing the section was in this size range. Since the total amount of material in suspension nearly doubled in this period, this represents nearly a four fold increase in the total volume of sand load in the 0.105 to 0.149 mm fraction. This suggests that this particular size fraction may be in a critical region that reacts immediately to increases in water viscosity by transporting all available material of this size existing in the bed of the stream. This of course is normally related to conditions governing the transport of silts and clays (wash load) but, when large variations in viscosity occur, may also relate to the smaller sand size fractions.

Changes noted in the mechanical analysis of the bed material appear to correlate with the above phenomena. The bed sample analysis shown on the same charts indicate that in the warmer periods, 6 to 10 percent of the bed was composed of material in the 0.105 to 0.149 mm size range, while in November only 2 percent was found to be in this range. This indicates a general coarsening of the surface layer of the stream bed as the finer material is picked up into the flow leaving the coarser size particles.

A detailed analysis of the individual point samples at a given location or vertical facilitates inspection of other parameters of interest. The slope of the concentration distribution relationship through the vertical, (Z), indicates how the various size fractions are distributed from the stream bed to the water surface. A small valve of "Z" indicates that the suspended material is evenly distributed throughout the vertical; whereas, a large value of "Z" indicates that sediment concentrations are larger near the bed than at the water surface. Plates 43 thru 49 show how this parameter varies with time at a given location. At most locations there appears to be very little change in this parameter, but at a few locations there appears to be an increase in "Z" for the larger sand size fractions as the load increases and the water temperature decreases.

Closely related to the above parameter is the actual sand concentration at an assumed reference depth in the flow. The concentrations at two reference depths are shown on Plates 43 thru 49. At $(d-y)/y=1.0$, (1/2 the depth), the most consistent change appears to be in the 0.105 to 0.149 mm size, where 3 out of 4 locations in the cross section show a steady increase in the concentration of this size fraction as the viscosity increases. Very little change can be noted in the concentrations of the smaller or larger size fractions.

At $(d-y)/y=100$, (at approximately 1/100 of the water depth from the bed), the concentrations are very erratic; however, increases in the concentrations of the coarser size fractions appear to be taking place as the viscosity increases.

Changes in the sediment carrying characteristics of the stream are also closely related to the type of bed form present in an alluvial stream. The shape of the bed forms have been shown to be influenced by such things as stream velocity, size of the material found on the bed of the stream, and suspended sediment load. A completely satisfactory method of relating all of these factors has not as yet been developed. Most prediction equations are good for streams which have similar hydraulic and sediment characteristics, but fail to encompass the entire range of conditions found in natural streams.

Plate 2[(2)] is an illustration of the bed forms existing near the centerline of the Missouri River navigation channel for several periods of the year. The profiles show that in early September the bed of the river was almost entirely composed of short steep bed forms, but with the continued passage of time, the formations gradually elongate until the profile is essentially flat. The profiles obtained in late October and early November indicate the bed was completely void of any surface irregularities. This complete change in the roughness characteristics of the channel is further reflected by a change in the average velocity of the stream and the resulting roughness coefficient (Plate 3).[(2)] The average velocity shows a steady increase from about 4.5 fps in August and September to about 5.25 fps in November, while Manning's "n" value decreases from 0.020 to 0.015.

Plate A of this report shows a relationship originally suggested by Einstein and Barbarossa[(7)] which relates the ability of a stream to transport material as bed load to the relative form roughness of the channel. The two dimensionless parameters used in this graph include measures of the hydraulic, sediment, and roughness characteristics of an alluvial stream. They include a measure of the bed grain size, the mass density of the bed material, the water surface slope, a measure of the relative roughness of the bed material, the stream velocity, and a measure of the form roughness of the channel. The plot shows how data from this investigation relates to the curve suggested by Einstein and Barbarossa for natural rivers. An excellent functional relationship is apparent. Water temperature is shown on the graph as a third variable, and it appears to be closely related to the location of points along the curve. This is in complete agreement with all the field measurements obtained as a part of this investigation. Other functions which include a measure of the viscosity also verify this finding.

DISCUSSION

The investigation exemplifies the importance of recognizing the close interrelationship that exists between the various parameters affecting sediment transport, resistance to flow, and bed roughness forms. This interaction of variables can be of extreme practical significance in streams like the Missouri, where it is desirable to not only predict changes in the sediment load, but also predict the unobstructed depth available in the navigation portion of the channel. However, the study also suggests other important considerations that should receive attention when developing data collection programs and establishing data reduction techniques.

The investigator must always be keenly aware of the many variables that are involved, and adapt his measurement program around those that are pertinent to his stream. For example, the location at which suspended samples are obtained relative to the position of major bars and dunes on the bed of a stream could very significantly affect the sample results. Other parameters such as fluctuations in the stream velocity, slope, bed grain size, and water temperature may also necessitate modifications to the sampling procedures.

A real danger lies in the fact that field data is many times obtained over short time intervals in isolated locations, but is used in the analysis as if it represented long term values applicable over the entire cross section. Many of the parameters known and verified as being related to the problem tend to be overlooked in the analysis, and only minimal effort is expended in the field to learn the true magnitude of the fluctuation of a given parameter. In some rivers it may be much more meaningful to concentrate the data collection program in selected reaches for a sufficient length of time to properly define the variables, rather than being tied to a system which obtains measurements at predetermined time intervals at fixed locations regardless of the river conditions. The entire data collection program must be built around a program which measures those parameters known to have a significant influence on the final result.

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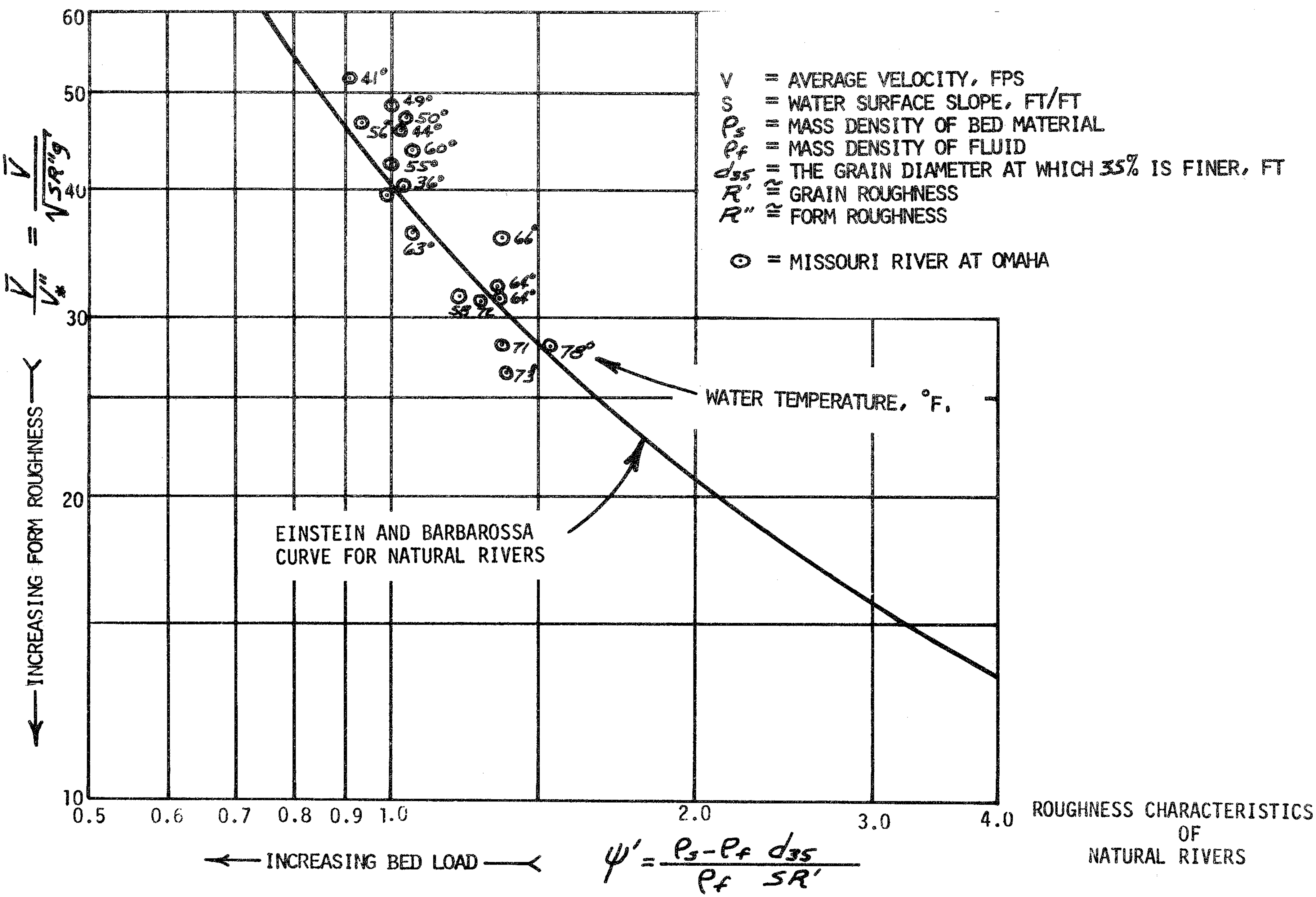

ROUGHNESS CHARACTERISTICS
OF
NATURAL RIVERS

PLATE A

INSIGHTS GAINED FROM RIVER SEDIMENTATION MODELS

by

John J. Franco[1]

Introduction

The movement of sediment in alluvial streams presents many problems that have to be considered in the development and improvement of these streams for navigation and flood control. The Corps of Engineers is vitally interested in these problems since they affect its principal civil works mission. In spite of the efforts and millions of dollars expended by this country and other countries throughout the world, little progress has been made toward a practical solution of many of the problems concerned with the effects of sedimentation. Because of the complex nature of these streams and the many related factors involved in their development processes, few basic principles have been developed that can be used by the design engineer in the solution of many of these problems or in the development of plans for the improvement of troublesome reaches.

Sedimentation problems will be encountered in any stream where there is a sizable movement of sediment into or within that stream. This movement of sediment and deposition within critical areas can affect channel depth, width, and channel alignment and often affects the operation and use of facilities and structures such as locks, harbor and docking facilities, hydroplants, sewage disposal, water intakes, etc.

The ASCE Tast Committee on Regulation and Stabilization of Rivers published a paper in 1965 entitled "Channel Stabilization of Alluvial Rivers." This paper summarized information contained in various papers on the subject in the hope that analysis of the data presented would lead to the establishment of certain guides that might improve and advance the profession. It was brought out in the paper that, because of the complex nature of these streams, the type of channel regulation and stabilization works used is still a matter of experience and general judgment. Unfortunately, this is essentially correct and many of the plans and structures adopted have been complete failures or have been ineffective in producing the desired results. The lack of significant progress in the science of river engineering could be attributed mostly to limited funds and to the reluctance on the part of many responsible engineers to support general research. Engineers who recognize and appreciate

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the need for the development of new procedures and methods too often feel that experiments should be conducted on the river itself. This type of experimenting is always costly, usually not properly evaluated or documented, and generally inconclusive. The development of generally applicable conclusive. The development of generally applicable conclusions from the evaluation of field data is difficult because of the differences in the characteristics of various streams, differences and variations in flow conditions, and irregularities in stream geometry even from one reach to the next of the same stream.

Laboratory Investigations

Funds for the development and maintenance of navigable streams make up a large portion of the Corps of Engineers annual budget; therefore, it is important that available resources be used with maximum effectiveness, which indicates the need for a continuing research program for the development of new and better design principles, construction methods, and maintenance techniques. The ASCE Task Committee on Regulation and Stabilization of Rivers indicated that valuable data can be obtained from model studies. However, it also stated that engineers charged with the responsibility of river control works hold that the solutions to problems remaining unsolved must be sought principally in the river or stream under study. Because of this feeling, many problems have never been resolved and trial-and-error methods have continued. Engineers have been experimenting with some rivers for several decades with only limited success. The present state of knowledge would indicate that few lessons have been learned from this type of costly experimentation.

Laboratory investigations have contributed to a better understanding of sedimentation processes and have led to the development of some of the theories involved which could not have been readily accomplished in natural streams. There has been considerable research and much has been written on the subject of sedimentation in general, and a considerable amount of effort has been put into the development of sedimentation formulas. In spite of the volumes of literature available on the subject of sedimentation, very little information is available on the control of sediment movement in natural streams. Most of what has been written is too general to be of much use to the design engineer in the development of practical solutions to many river problems. Because of the difficulties and costs involved in experimenting with the actual river and impracticability of comparing the effectiveness of various concepts and designs under the same conditions, it is the writer's opinion that the development of new principles and procedures will have to depend to a considerable extent on model investigations coordinated with results in the field.

The Waterways Experiment Station has been concerned with the solution of sedimentation problems since it started operation. Most of the studies have been concerned with the solution of specific problems in unusually troublesome or unstable reaches. Other studies, relatively few in comparison to the need, have been in the nature of general investigations including participation in field investigations and analysis and evaluation of field data. These studies have led to a better understanding of some of the processes involved in river development and of reasons for success or failure of some of the structures and procedures, and to the development of some basic principles that should be used in the design of structures for maximum effectiveness at least cost. Some of the studies and results developed in these studies are summarized herein.

Arkansas River Development

One of the largest projects undertaken by the Corps in recent years is the development of the Arkansas River for navigation. The Arkansas River carries a tremendous sediment load and in its natural state was a wide, shallow stream with steep slopes. The low-water channel with controlling depths of not more than 1 or 2 ft meandered within its banks and within the floodplain. The river is being canalized by means of a series of locks and dams, and a comprehensive system of channel rectification and training structures. The dams are gated structures with the gate sills near the bed of the river. The development of the Arkansas River involves the solution of many sedimentation problems, some of which have not been encountered on other streams. During the planning for this project, questions were raised as to the effects of the locks and dams on the movement of sediment, operation of the gates, best location for the structures, elevation of the sill with respect to the existing bed, and the bed that could be expected to result from construction of the improvements contemplated.

During the early stages of planning for the project, two idealized types of model studies were made to determine the nature and magnitude of the problems that might be encountered. Since that time, a number of studies to obtain information that would assist the design engineer in the solution of many general and specific problems and in the development of plans connected with the project have been completed.

Movement of Sediment Between Locks and Dams

The movement of sediment within a pool on a canalized stream depends on the river discharge, operation of the spillway gates, location within the pool, etc. When discharges are sufficient to maintain a normal or higher pool level at the dam, open river conditions prevail and the gates are in the raised position (fig. 1). With this condition the water-surface slope

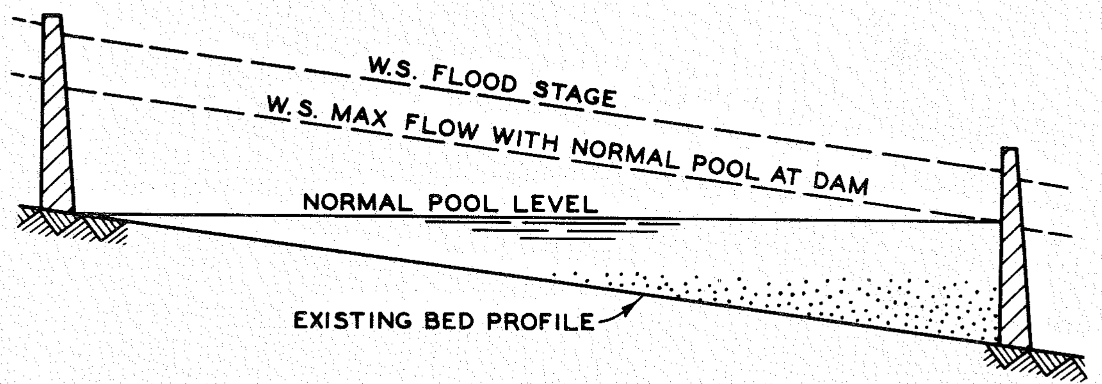

Fig. 1. Variation in water-surface slope between dams

is generally parallel to the bed. As the discharge decreases, the gates are closed to maintain a minimum pool level at the dam. Velocities are reduced because of the backwater effect of the gates and deposition starts near the dam. The point of deposition moves progressively upstream as the discharge continues to decrease and the amount of gate closure is increased. While sediment is being deposited in the lower reach of the pool, movement of material continues in the upper reach. During controlled river flows, essentially a degrading and aggrading stream exists between dams. When the discharge is again increased and the gates are opened, the rate of sediment movement is greater near the dam than in the rest of the pool because of the deposition. How much of this material is moved out would depend on the magnitude and duration of flow.

Although operation of the gates is used to maintain minimum pool level some 20 ft or more above the natural low-water level, depths of 9 ft required for navigation are not necessarily obtained in the upper lock approach. The effects of currents on navigation must be considered in location of the lock and dam structures with respect to channel alignment and training structures used. A model of a lock and dam partially drained to outline the channel and show the training structures used to develop depths in the approach to the lock is shown in fig. 2. The shoaling in the crossing upstream can also be seen in this figure. Because of the long straight reach, the channel tended to meander; the lock which obstructed flow along that side increased this tendency. Operation of the gates could do little to offset this tendency since most of the movement of sediment occurs when the gates are open and essentially open river conditions prevail.

Shoaling in Lower Lock Approach

Shoaling in the lower lock approach is a Corps-wide problem encountered at most of the structures in streams carrying sediment. The problem in the Arkansas River will be greater than that experienced at locks in other streams such as the upper Mississippi and Ohio Rivers because of the sediment load. The channel is wider at the dam than it is farther downstream to provide for the effect of the piers, elevation of the gate sill, and capacity of the spillway to pass flood flows with little effect on stages (fig. 3). Some of the sediment moving through the dam during higher flows is deposited in the wide area and moves downstream during low water because of the shallow depth and steep slope. Most of the material moving along the lock side of the channel is carried into and deposited in the approach channel where velocities are reduced by the increase in width and depth landward of the lock wall; some of this material is also moved into the approach by eddy currents.

Unless something is done about this problem, dredging will have to be almost continuous in the Arkansas River since any shoaling landward of the end of the wall would make it difficult for a tow to approach the wall and become aligned for entrance into the lock. Model studies have indicated that a wing dike placed at a slight angle to the alignment of the lock wall would reduce the dredging frequency and in some cases the amount of dredging. The dike is designed to permit surface flow to move over the top and prevent the bottom sediment-laden currents from moving into the channel after passing the end of the dike. The dike should be high enough to prevent the movement of sediment over the top and low enough to permit enough flow over the top to prevent bottom currents from moving sediment into the channel. Some reduction in dredging could be accomplished by operation of the lock gates. Closing of the gates on the dam away from the lock, insofar as conditions will permit, would cause some of the material scoured downstream of the open gates to be deposited behind the closed gates. Limited studies in the model have indicated that bypassing flow into the lower lock approach through filling culverts, through the lock or by special bypass channel would be impractical because of the amount of water required to produce scouring velocities.

Investigation of Dikes and Dike Systems

The development of the authorized 12-ft channel in the Mississippi River between Cairo and Baton Rouge will require the construction of more than a

Fig. 2. Channel development upstream of a lock and dam. Water Surface lowered to 14 ft below normal pool

Fig. 3. Shoaling in lower lock approach with three wing dikes at end of lower guard wall

million linear feet of dikes at an average estimated cost of about $90 a foot. Although dikes have been used extensively throughout the world for many centuries, few basic principles have been developed that could be used as an aid in the scientific design of structures to meet specific requirements. A laboratory investigation undertaken within the last few years to determine some of the factors affecting the performance of dikes and dike systems has produced several new concepts for use in the design of these structures for the improvement of specific river reaches or in the solution of specific problems. This study has demonstrated why some recently constructed dike systems were not effective and how a much more effective system could have been used at a much lower cost. The effectiveness of dike systems depends on the effects of the systems on the movement of currents and sediment around and within the systems. The movement of currents within dike systems using different design principles in shown in fig. 4. The relative effectiveness of various dike systems is based on a rating system developed for the purpose, which considers such factors as increase in controlling depth, channel alignment, reduction in dredging required to obtain project dimensions, maximum scour at the end of the dike, and deposition within the dike system.

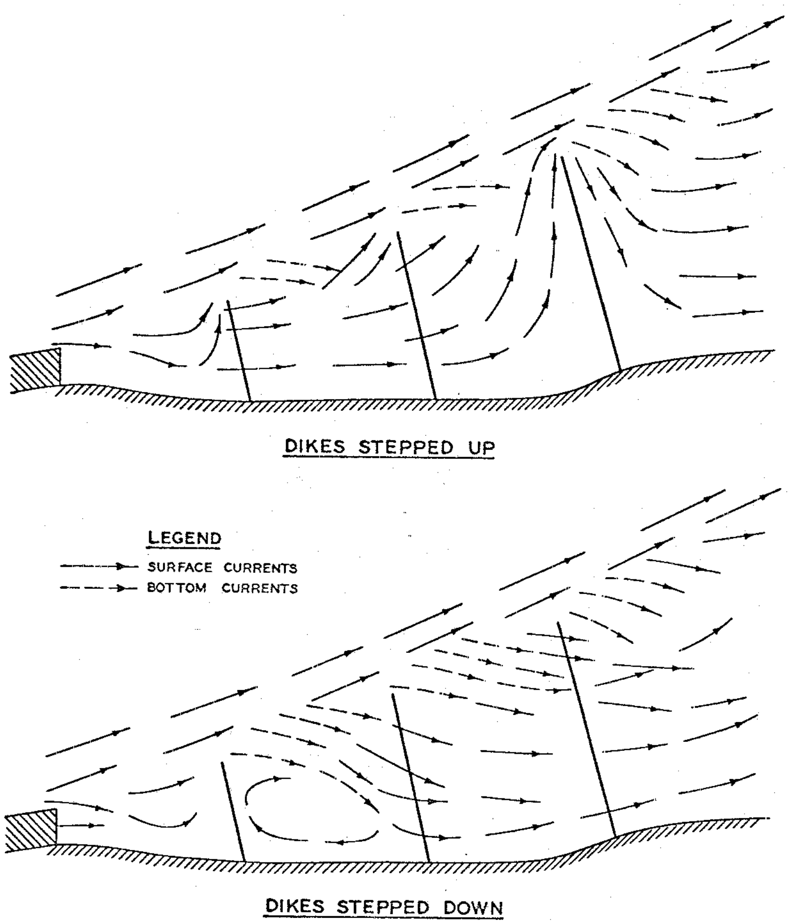

Fig. 4. Currents within dike systems

Fig. 5. shows the comparative effects of several dike design principles in reducing dredging or the amount of material forming the shoal. This type of comparison is made possible in the model since all systems were tested at the same location and subjected to the same flow hydrograph with stages varying from 5 to 40 ft above mean low water. The dike studies have been conducted by setting up typical problem reaches and studying the effectiveness of proposed designs in solving these problems. One of the problems submitted was concerned with the development of an effective method of closing a back or chute channel. When various designs of the conventional type submitted for

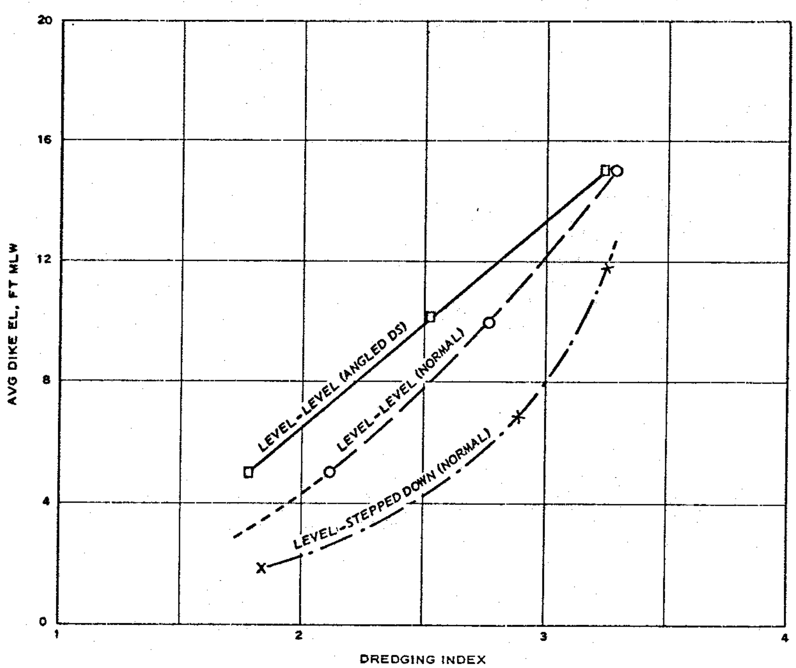

Fig. 5. Reduction in shoaling in thousands of cubic yards per foot of channel width

testing were indicated to be no more effective in the model than they were in the Mississippi River, the writer suggested the use of vane dikes. These dikes are short lengths of stone dikes 500 to 1000 ft long, spaced from 600 to 1000 ft apart, and angled about 10 degrees to the direction of flow. Rather than force the river out of the back channel with massive structures which would be difficult to maintain, vane dikes are placed in relatively shallow water and are designed to divert sediment into the back channel. These structures, currently being used in two districts on the Mississippi River, have been successful and relatively stable. These structures have wide application and have been used to control as much as 50 percent of the total river flow diverted from the main channel. Modification of a contract based on the use of these structures resulted in a savings of $500,000 at one installation. A savings of more than $1.7 million was documented by the Mississippi River Commission in one year based on the use of new principles developed.

Effect of Lateral Differential in Water Level on Sedimentation

In studies concerned with river bifurcation, the basic principle that a bypass channel will take a greater proportion of the total sediment load than the proportional total flow is well established. The reason for this is that the iertia of the faster-flowing surface water with its realtively lower sediment load will tend to be carried past the side channel, leaving more of the slower-moving bottom flow with its heavy sediment load to turn into the side channel. The principle could be stated simply as follows: When conditions exist that produce a lateral differential in water-surface elevation there will be a tendency for at least some of the total river flow to move toward the lower elevation; the lower-moving sediment-laden bottom currents can make the change in direction easier than the faster-moving surface currents and account for the greater movement of sediment toward the lower level. This principle is involved in many of the developments in alluvial streams including the development of point bars, movement of sediment around the end and behind dikes, shoaling in lock approaches, etc. In each case there is either a buildup of head on one side or a reduction in head caused by channel enlargement or flow diversion which results in a change in the normal direction of some of the streamflow.

A large number of bifurcated channels formed by sandbars or islands exist in the Mississippi River and other alluvial streams in addition to those formed by cutoffs. Most of the sediment moved into a side channel is deposited at the head of that channel, producing a shoal area across the entrance which eventually reduces or eliminates the discharge into that channel during the lower river stages. When deposition occurs at the upper end, the sediment-free flow moving downstream could cause scour and deepening of the side channel downstream of the entrance, and in a number of cases serious bank caving has occurred. The amount of sediment diverted and the effect of shoaling would depend upon the relative size, shape, and angle of the entrance with respect to the direction of flow from upstream and the relative lengths of the two channels.

Open River Problems

Changes in the discharge and stages produce changes in currents and in the movement of sediment in natural streams and render the application or development of design principles extremely difficult. Model and field investigations have indicated how channel depths and configurations can be altered with change in stages. The movement of sediment in one reach can be considerably higher than in a reach just downstream during low flows and considerably lower during high flows.

Most problems with regard to development of a channel within an alluvial river channel will be encountered in long, straight reaches, in bends having large radii of curvature, and in crossings. In long, straight reaches and long flat bends, the channel will tend to meander; and channel depths will tend to be uniform between the controlled channel limits. Sand waves moving through these reaches at high flows could form a shoal, with a rapidly falling river causing a deeper channel to develop on the other side of the channel. Although such formations appear to be random, they usually occur at specific locations where there is a tendency for the channel to meander.

Alignment of the channel and depths over crossings depend to a considerable extent on flow conditions and alignment of the bend upstream, and to some extent on the alignment of the bend downstream. Maintaining a channel in crossings will be more troublesome when regulating structures on the concave side of the bend upstream are not carried far enough downstream to prevent dispersion of most of the higher flows and when the crossing to the next bend is long. Extending the regulating works in the bend upstream toward the crossing improves the alignment and depth of channel over the crossing and improves flow in the next bend downstream.

The changes in low-water slope profiles can be attributed in most cases to the relative movement of sediment in successive reaches. When the low-water slope is higher than the average, it is generally an indication that more sediment was moved into that reach from upstream during the higher flows than could be moved through the reach during the same flows. The slope of any stream having a movable bed whether it is a canal, flume, or river can be changed by changing the amount of sediment introduced without changing any of the other factors affecting flow in the stream. A sinuous channel can move much more sediment for the same average velocity than can be moved in a straight channel. For this reason, straight reaches following a sinuous reach will tend to be troublesome and unstable. The low-water slope is usually an indication of what has happended during the higher flows when the bulk of the total sediment is moved. Because of the difference in the capacity to move sediment, low-water slopes will tend to be higher in troublesome and unstable reaches and in crossings. In analyzing the capacity of various reaches of a stream to move sediment, sinuosity of the channel should be considered.

Conclusion

The Corps is concerned with many sediment problems in the development and maintenance of natural streams, particularly for navigation and flood control. These problems will be encountered in any stream where there is sediment moving into or within that stream. Shoaling problems can be expected at mouths of tributary streams; in crossings; in long, straight reaches or in long, flat bends where the channel tends to be unstable; in lock approaches; and in the entrance to slack-water harbors. Maintenance dredging is only a temporary solution since in most cases it has to be repeated periodically.

A knowledge of the amount of sediment moving in a natural stream is important; but in order to be able to develop solutions to many of these problems, it is more important to know the source of the sediment contributing to the problem, factors affecting its movement, and what can be done to eliminate the problem.

The design of structures for the improvement or development of a stream should be based on the general characteristics of that stream and the effects of the proposed structures on currents and on the movement of sediments for all significant flows. In navigable streams the effects of structures on conditions affecting navigation must also be considered.

Experiments with the river can be expensive and inconclusive because of the many variations that exist in alluvial streams. Field observation and analysis of field data require considerable time and effort and can be frustrating because of the many inconsistencies that can be expected, owing to the irregularities in geometry, turbulence, pulsating currents, and constantly changing beds.

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SEDIMENTATION STUDIES FOR ROBERT S. KERR LOCK AND DAM ARKANSAS RIVER BASIN

by

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INTRODUCTION

The Arkansas River multiple-purpose project, authorized by Congress in 1946, provides primarily for navigation, hydroelectric power, and flood control. The 450 mile navigation route from the Mississippi River to Catoosa, Oklahoma, will consist of a navigation channel with a minimum depth of 9 feet and a minimum width ranging from 150 to 300 feet, a series of 17 locks and dams, and bank stabilization and channel rectification works. Flood control storage of about 6,000,000 acre-feet will be provided by the following seven upstream reservoirs in Oklahoma: Tenkiller Ferry, Eufaula, Pensacola, Keystone, Oologah, Markham Ferry and Fort Gibson. Hydroelectric power will be generated at the above reservoirs, and also at the following four locks and dams: Dardanelle, Ozark, Robert S. Kerr, and Webbers Falls. The plan for the Arkansas River multiple-purpose project is shown on plate 1.

The solution of major sedimentation problems played an important role in the planning, design, and construction of the Arkansas River multiple-purpose project. Investigations included the determination of natural and modified sediment loads, sediment deposits in channels and reservoirs and degradation below dams, and the prediction of modified channel regime.

This paper will review sedimentation studies conducted for one individual reservoir project, Robert S. Kerr Lock and Dam, of the Arkansas River multiple-purpose project. The sedimentation studies will be summarized and several techniques used for estimating degradation below the dam will be described.

GENERAL

Robert S. Kerr Lock and Dam

The Robert S. Kerr Lock and Dam is presently being constructed on the Arkansas River at river mile 395.4, about 8 miles south of Sallisaw, Oklahoma. The construction of the project is near completion and closure is tentatively scheduled for the latter part of 1970. The project will have a dead storage capacity of 414,100 acre-feet with the navigation pool level at elevation 458, and an additional storage capacity of 79,500 acre-feet for power with the top of power pool at elevation 460. The power plant will contain four generating units with a total installed capacity of 110,000 kilowatts. The lock will provide a lift of 48 feet for navigation.

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The project was originally designated as the Short Mountain project. Congress changed the name of the project as a memorial to the late Robert S. Kerr, United States Senator from Oklahoma.

Sedimentation Study Objectives

In 1962, Tulsa District made a final analysis of the sediment problems related to the design of the Robert S. Kerr project. Prior investigations were reviewed and additional sedimentation studies were conducted. Degradation studies were made to determine future tailwater conditions required for the design of the lock, power plant and spillway. Reservoir deposition studies were made to determine future sediment profiles and corresponding envelope curves of backwater effects to be used as a basis to establish design criteria for relocation of facilities and guidelines for land aquisition. The final sedimentation analysis is discussed further herein.

Bank Stabilization and Channel Rectification Program

The Arkansas River channel will be altered considerably by construction of bank stabilization and channel rectification works between Robert S. Kerr and the head of Ozark Lock and Dam (Fort Smith, Arkansas). The stabilization works will realign the river channel to provide sufficiently long radii bends for the anticipated tows and a hydraulically stable alignment that can be maintained with bank revetment. Realignment of the channel is dependent on inducing the river to scour or deposit sediment in designated places. The movement of the channel is accomplished by deposition of sediment in the slackened water near stone or pile dikes built from the banks out into the channel. The dikes are extended, and additional ones are built as the deposits accumulate until the channel reaches the desired alignment.

Construction of the stabilization works on the Arkansas River between Robert S. Kerr and Lock and Dam No. 14 was initiated in 1960 and is now essentially complete. Construction began in 1952 and was essentially completed in 1959 for the reach downstream from Lock and Dam No. 14 to Fort Smith, Arkansas. Periodic hydrographic surveys (1952, 1954, 1955 and 1958) of the channel below Lock and Dam No. 14 indicate the changes that occur to the channel from the construction of stabilization works.

The realigned channel width will be 1,000 feet. The pre-project channel widths varied from 1,300 to 2,600 feet. The realigned channel length between Robert S. Kerr and Lock and Dam No. 14 of 16.9 miles compares with channel lengths of 20.3 miles in 1940 and 18.6 miles in 1960.

The channel below Robert S. Kerr will be further modified near time of closure when the proposed navigation channel with a 250-foot bottom width and 4:1 side slopes will be dredged to a bed elevation of 400.0 to obtain the twelve feet of depth required for navigation. The bottom of the proposed dredged channel is shown on plate 2.

SEDIMENT LOAD AND SEDIMENT DEPOSITION

Sediment terms used herein are defined as follows: The "sediment load" of a stream at a specified location consists of all the sediment particles moved by the currents of that stream for a specified time period. The "suspended load" is that portion of the sediment load transported in suspension by currents of the stream. The remaining portion of the load is referred to as "bedload," and is defined as particles which move as a layer near the stream bed by rolling or bouncing. Sediment load may also be divided into "wash load" and "bed material load." Wash load is that part of the load which is not present in the stream bed in significant quantities and usually consists of silt and clay particles transported in suspension. The bed material load is composed of particles found in relative abundance in the stream bed and may be transported as suspended bed material load or bedload. The term "bed material" signifies the material in the stream bed.

At the Robert S. Kerr dam site, the natural suspended sediment load and corresponding suspended bed material load and wash load were estimated in earlier studies 1/ to be about 95, 22, and 73 million tons per year, respectively. It was considered that these loads represented natural conditions with no reservoir projects in place in the Arkansas River basin. The loads were estimated from an analysis of sediment and flow records collected during the 15-year period 1939-1953. For this period, the average stream flow was determined to be about 34,000 cfs.

These earlier studies indicated that the natural sediment load at the dam site would be reduced significantly by the authorized upstream reservoir projects. The modified sediment load and corresponding bed material load and wash load were estimated to be about 11, 2, and 9 million tons per year, respectively. Furthermore, it was estimated that 44 percent of the modified sediment load flowing into the Robert S. Kerr reservoir would be deposited in a 50-year period, resulting in a reduction in storage capacity of 34 percent (171,000 acre-feet). Table 1 shows data on the estimated modified sediment loads for the 50-year period.

The deposits shown in table 1 were computed by the detention-time method. Detention time is defined as the ratio of the reservoir storage capacity to the inflow discharge at any given time. Curves of detention time versus percent load deposited (shown on plate 3) and modified wash (silt-clay) load and bed material load curves were used to determine the amount of sediment deposited.

The detention-time method was also used in 1962 to determine the distribution of sediment deposits within the reservoir area. A year-by-year study was made for a 50-year period. The resulting 50-year sediment profile is shown on plate 4. For this study, the reservoir was subdivided into six reaches and the storage capacity, including the storage under the backwater profile, of each reach was calculated. Backwater computations were made for initial conditions and were repeated at the end of each 5-year period, and the storage capacity of each reach was adjusted accordingly. Detention times were computed based on the accumulative upstream storage capacities. The differences in sediment deposited at the ends of a reach were assumed to be the amount of sediment deposited in that reach.

DEGRADATION

Degradation Factors

There are certain important factors to consider in evaluating degradation below a dam constructed on an alluvial stream. For appreciable degradation to occur, the stream must carry a relatively large bed material load under natural conditions as this is a measure of its ability to transport bed material. The material in the bed to some considerable depth must be of a size transported by the river, and the reservoir impoundment created by the dam must have sufficient detention time to trap most of the bed material load so that the outflow is essentially free of bed-sized material. Furthermore, if a major portion of the natural load was transported by high flows, then some of these high flows must also continue to take place under modified conditions for degradation to occur. Segregation of gravel may armor the bed and retard degradation. If the banks are not protected, the river may attack the banks rather than the bed for its source of sediment supply. All of the above factors will influence the rate of degradation below Robert S. Kerr.

Bed Material

The stream bed of the Arkansas River between Robert S. Kerr and Lock and Dam No. 14 is composed of sand and gravel overlying bed-rock to depths of from a few feet to about 20 feet. Core hole borings to bed-rock were made to obtain bed material samples for a grain size analysis of the stream bed, and to determine top of bed-rock elevations.

Top of bed-rock elevations were obtained in 1959 from core hole borings spaced 500 feet apart in the proposed navigation route between Robert S. Kerr and Fort Smith, Arkansas. Portions of the top of bed-rock profile and the assumed thalweg based on degradation to bed-rock control points are shown on plate 2.

Bed material samples were retained from eight of the above core hole borings between Robert S. Kerr and Lock and Dam No. 14. The location of these holes, which are spaced about 2 miles apart, are shown on plate 2. Bed material samples were also retained from eleven core hole borings in 1959 and nine core hole borings in 1946, which were taken outside the limits of the proposed 1,000 foot wide stabilized channel.

Mechanical analysis curves on the bed material samples for various depth intervals were computed and analyzed. These curves indicated a higher percentage of finer bed material in the lower end of the reach. Therefore, the river was subdivided into two reaches, with reach No. 1 upstream of navigation mile 321.5 and reach No. 2 downstream. The estimated average grain size distribution of the bed material for each reach is illustrated by the curves shown on plate 5.

It was estimated that 15,800 acre-feet (32 million tons at 93 pounds per cubic foot) of bed material would be available for transport by river flows within the limits of the 1000 foot wide stabilized channel. This estimate was based on computing the quantity of bed material between the apparent bed-rock control thalweg and the 1961 stream bed thalweg adjusted for the proposed dredging of the navigation channel after closure to elevation 400. Table 2 shows the estimated bed material quantities for the individual grain sizes in reaches Nos. 1 and 2.

Computation Procedures

Three methods were used for computing the rate of degradation on the Arkansas River downstream from Robert S. Kerr. One method was based on the equation for bedload function as described by Dr. H. A. Einstein in the publication, "The Bedload Function for Sediment Transportation in Open Channel Flows" 2/. Another method was the procedure used by Little Rock District for computing degradation below Dardanelle Lock and Dam 1/. This method makes use of the Kalinske formula to compute bedload and relationships between natural suspended bed material load and discharge to compute the modified suspended bed material load. The third method was based on the determination of the thickness of an armor layer and the depth of scour at which this layer would form.

Principal factors considered in the application of these methods was the composition of the bed material and its coarsening with time, magnitude of flows to be expected in the future, and channel flow characteristics such as shape of channel, flow depth, velocity, and slope.

Channel flow characteristics were established from backwater computations for various flow conditions using Manning's "n" values ranging from .020 to .035 for the channel and from .05 to .06 for the overbank areas. The various flow conditions were based on estimating a thalweg profile which changes with time, and using a typical channel section. The ultimate thalweg profile was assumed to be the rock control thalweg shown on plate 2. As the riverbed degrades, it was assumed that the typical channel section would not change.

The typical channel section was determined by averaging channel cross sections obtained from the 1958 hydrographic survey of the Arkansas River channel between Lock and Dam No. 14 and Fort Smith, Arkansas. The use of a typical channel section simplified the computations, and was considered to be warranted due to uncertainties about future changes which might occur in the shape of the channel sections. The changes would vary widely with local irregularities in bed resistance. River crossings tend to aggrade during high flow periods whereas riverbends tend to erode. The reverse of this situation occurs during low flow periods.

Cycles of high and low flow periods are characteristic of the Arkansas River; therefore, the degradation of the stream bed will be intermittent rather than at a uniform rate. However, in this study a uniform rate was used to simplify the computations. The regulated flow duration relationship derived in prior studies 3/ for the Sallisaw stream gaging station was used. The regulated flow duration curve is based on 35 years, 1923 through 1957, of computed daily flows as regulated by the authorized upstream reservoir system.

Estimated backwater effects from Lock and Dam No. 14 would reduce the normal water surface slope for flows less than 90,000 cfs to the extent that the sediment carrying capacity of these flows could be neglected. Regulated flows will equal or exceed 90,000 cfs 10 percent of the time. From backwater computations, it was determined for flows above 90,000 cfs that an average water surface slope of .000265 in reach No. 2 and .00023 in reach No. 1 would prevail initially. As the stream bed is scoured, the average slope in reach No. 2 would decrease to .00023 and in reach No. 1 would increase to .000265. The average slope in reach No. 1 would decrease to .00023 upon degradation to bed-rock control.

Degradation Limited by Armoring

An indication of how much the riverbed might degrade was obtained from a simplified procedure based on determining the grain size of bed material which cannot be transported in any appreciable quantity and assuming that degradation would cease when a layer of "non-moving" sediment equal to this grain size "shingles" or "armors" the bed. The continuous layer would act as a protective pavement thus preventing any further motion of the finer particles underneath.

Formulas suggested by Dr. H. A. Einstein 4/ for computing grain sizes that would not move under given flow conditions were used, and the results are as follows:

16 millimeter	- 50,000 cfs
32 millimeter	- 130,000 cfs
48 millimeter	- 430,000 cfs

Under future regulated conditions, flows of 50,000 cfs and 130,000 cfs would be equalled or exceeded 17 and 4.6 percent of the time, respectively. A flow of 430,000 cfs is equivalent to the peak of the regulated 50-year flood. This indicated that movement of the coarse gravel sizes (16 to 32 m.m.) was to be expected and that only slight movement of the very coarse gravel sizes (32 to 64 m.m.) at the lowest limit of the size range would occur.

To form an armor layer equal to the diameter of 32 m.m. (1.3 inches) particle size would require the bed to scour a depth of 9.5 feet. If the flat side of an elongated 32 m.m. particle size is one-half the major axis, then it would be necessary for the bed to scour to a depth of 4.8 feet. It was concluded that degradation ranging from 5 to 10 feet could be expected. This procedure gives no indication of the rate at which degradation might occur.

Little Rock District Method

The same procedures used by Little Rock District for computing degradation below Dardanelle Lock and Dam were used to compute the rate of degradation below Robert S. Kerr for reach No. 1 in a year-by-year study for a 25-year period. A water surface slope of .00023 was used, and a six-inch lift was selected to obtain an indication of how the bed material near the surface would coarsen with time and possibly armor.

The Kalinske formula was used to obtain the bedload. The average annual bedload results for the 25-year period are shown in table 3, and for comparison purposes the bedload for year No. 1 is included to show how the bedload increases as the bed material at the surface coarsens, especially in the gravel grain sizes.

The suspended bed material load for individual grain sizes was assumed to be equal to the modified load shown in table 4. The values in the table were estimated by using the regulated flow duration curve for Sallisaw and curves of discharge versus suspended bed material load.

The bed material load would initially exceed the bed material available for transport in the very fine sand and fine sand grain sizes, and by year 25 would exceed the bed material available for transport for all the sand sizes except very coarse sand. Initially there would be 1.5 inches of gravel, and at the end of 25 years there would be 4.4 inches of gravel in the six-inch lift. Shown on plate 5 are a series of curves which show the composition of the bed material for the six-inch lift for the various years as indicated. During this 25-year period a total of 7,100,000 tons of bed material would be removed and the bed would be lowered an average of about 3.5 feet. The bedload accounted for 98 percent of the bed material removed, because the movement of the sand grain sizes in suspension was limited by the amount of bed material available for transport in the six-inch lift. If a larger lift had been selected then more of the bed material would have been removed.

The results give an indication of how the bed material near the surface would coarsen as the degradation process progresses. The question arises whether the presence of 4.4 inches of gravel near the bed surface at the end of 25 years would shield the finer sized particles below. The regulated flows are capable of moving all the gravel sizes except very coarse gravel, which would occupy 7.5 percent of the six-inch lift at the end of 25 years. The lower size limit of very coarse gravel is 32 m.m. (1.3 inches). It would require 21.6 percent very coarse gravel in the six-inch lift to form a layer equal to the diameter of a 32 m.m. sized particle. It may be that the grain size of coarse gravel is capable of armoring the bed, since the rate of removal is small and there is a sufficient amount of this size present to form a layer of 2.3 inch thickness at the end of the 25-year period.

The degradation results for the 25-year study for reach No. 1 are considered to be low, because water surface slopes higher than .00023 are to be expected and the movement of the sand grain sizes was limited by the selection of a six-inch lift.

Einstein Bedload Function

The Einstein formula for bed material load was computed for water surface slopes of .000265 and .00023. The grain size distribution shown on plate 5 for reach No. 1 and reach No. 2 was used. The bed material load rate was computed (no bank friction considered) and the modified flows above 90,000 cfs were applied to obtain the bed material load. The results are shown in table 5.

The reduction of the bed material load due to variations in slope are shown in the two columns for reach No. 1. The change in bed material load due to composition of the bed is reflected in the latter two columns. For reach No. 1 it is considered that the bed material loads computed for the two slopes bracket the amount of bed material that would be carried by the river flows.

Tailwater

Tailwater rating curves for designated channel conditions are shown on plate 6. The tailwater rating curves were obtained from extensive backwater computations. The tailwater rating curve at time of closure shows the estimated effect of further development of the bank stabilization and channel rectification program. Immediately after closure, dredging of the 250 foot wide navigation channel will be accomplished which will lower the tailwater rating curve about two feet. This tailwater rating curve represents initial conditions. The 50-year weighted tailwater rating curve represents conditions with the typical channel section eroded to the apparent bed-rock control thalweg. The tailwater curve shown for maximum degradation represents conditions, assuming that the typical channel section widens considerably at the bed-rock control thalweg.

The difference of about two feet between the 50-year weighted and maximum degradation tailwater rating curves shows the effect of using a wider channel section at the lower depths of flow. If a typical channel section had been selected with wider widths at lower depths of flow then the 50-year weighted tailwater rating curve would occur before the channel bed is eroded to the bed-rock control thalweg.

Conclusion

It was concluded from the degradation study that the presence of coarse gravel in the bed will hinder the rate of degradation, and that in certain areas coarse gravel combined with the relatively nonmoving very coarse gravel will armor the bed. In those areas where a gravel control or armoring might occur, selective dredging may be necessary to expose the bed to flow which would cause an increase in the rate of degradation. This may have to be repeated several times to lower the bed to desired levels that would correspond with the 50-year weighted tailwater rating curve. It was not anticipated that major dredging would be necessary to lower the tailwater rating curve to the design level.

SUMMARY

In reviewing the final analysis made in 1962 on sediment problems related to the design of the Robert S. Kerr project, certain questions came to the writer's attention. If this study were to be repeated now, how would the study be conducted? Are there new or better techniques available for accomplishing the study objectives? Would the results be substantially different?

It is recognized that the use of automatic data processing equipment would facilitate the study and thus improve the accuracy of results somewhat. For example, computer programs could be developed to perform daily or weekly routings to estimate degradation or reservoir deposition rates. The use of daily or weekly flows in lieu of a flow duration relationship would improve the accuracy of results, although to what extent is not known at this time.

The availability of more data might improve the accuracy of the study. For instance, the typical channel section adopted for the 1962 study could be adjusted, if required, based on more recent hydrographic surveys of the stabilized 1000 foot wide channel below Robert S. Kerr.

It is hoped that the participants at this seminar will provide insight on new technology in the field of sedimentation, especially, for computing the distribution of sediment deposits, rate of degradation and depths of scour at which armoring would retard the degradation process.

ACKNOWLEDGMENT

The writer wishes to express his thanks to Mr. Bruce Cox, Mr. Fred Becker, and Mr. Cecil Courcier of Tulsa District for their assistance, guidance and cooperation. Mr. Becker was most helpful in furnishing the data and plates used in this report. Mr. Courcier and the writer performed the 1962 sedimentation analysis under the direction and guidance of Mr. Cox.

TABLE 1

MODIFIED SEDIMENT LOADS AT SHORT MOUNTAIN

Year	Inflow: Bed material load	Inflow: Silt-clay load	Inflow: Total	Deposit: Bed material load	Deposit: Silt-clay load	Deposit: Total	Outflow: Bed material load	Outflow: Silt-clay load	Outflow: Total
				(1,000,000 tons/year)					
1	19.74	13.60	33.34	0	0	0	19.74	13.60	33.34
2	15.49	13.60	29.09	0	0	0	15.49	13.60	29.09
3	5.40	10.40	15.80	4.98	4.91	9.89	.42	5.49	5.91
4	5.37	10.40	15.77	4.93	4.84	9.77	.44	5.56	6.00
5	5.36	10.40	15.76	4.92	4.78	9.70	.44	5.62	6.06
6	1.76	8.78	10.54	1.48	3.66	5.14	.28	5.12	5.40
7	1.78	8.80	10.58	1.51	3.64	5.15	.27	5.16	5.43
8	1.79	8.82	10.61	1.51	3.62	5.13	.28	5.20	5.48
9	1.44	8.85	10.29	1.22	3.61	4.83	.22	5.24	5.46
10	1.41	8.87	10.28	1.20	3.59	4.79	.21	5.28	5.49
11	1.44	8.87	10.31	1.22	3.56	4.78	.22	5.31	5.53
12	1.44	8.91	10.35	1.23	3.55	4.78	.21	5.36	5.57
13	1.46	8.93	10.39	1.24	3.52	4.76	.22	5.41	5.63
14	1.49	8.95	10.44	1.26	3.50	4.76	.23	5.45	5.68
15	1.51	8.96	10.47	1.28	3.49	4.77	.23	5.47	5.70
16	1.52	8.98	10.50	1.29	3.47	4.76	.23	5.51	5.74
17	1.55	9.00	10.55	1.31	3.46	4.77	.24	5.54	5.78
18	1.57	9.02	10.59	1.32	3.44	4.76	.25	5.58	5.83
19	1.58	9.03	10.61	1.34	3.40	4.74	.24	5.63	5.87
20	1.60	9.05	10.65	1.35	3.38	4.73	.25	5.67	5.92
21	1.62	9.07	10.69	1.37	3.35	4.72	.25	5.72	5.97
22	1.64	9.08	10.72	1.39	3.34	4.73	.25	5.74	5.99
23	1.66	9.09	10.75	1.40	3.32	4.72	.26	5.77	6.03
24	1.68	9.12	10.80	1.41	3.30	4.71	.27	5.82	6.09
25	1.70	9.14	10.84	1.43	3.28	4.71	.27	5.86	6.13
30	1.80	9.21	11.01	1.50	3.14	4.64	.30	6.07	6.37
35	1.91	9.28	11.19	1.57	3.02	4.59	.34	6.26	6.60
40	1.99	9.33	11.32	1.62	2.90	4.52	.37	6.43	6.80
45	2.08	9.38	11.46	1.67	2.79	4.46	.41	6.59	7.00
50	2.16	9.44	11.60	1.72	2.66	4.38	.44	6.78	7.22

TABLE 2

BED MATERIAL QUANTITIES

Grain Size Classification	Reach No. 1 (tons)	Reach No. 2 (tons)	Total (tons)
Very coarse gravel	176,000	128,000	304,000
Coarse gravel	736,000	320,000	1,056,000
Medium gravel	944,000	400,000	1,344,000
Fine gravel	816,000	352,000	1,168,000
Very fine gravel	1,280,000	512,000	1,792,000
Very coarse sand	2,048,000	991,000	3,039,000
Coarse sand	3,856,000	3,887,000	7,743,000
Medium sand	3,536,000	5,935,000	9,471,000
Fine sand	1,696,000	2,835,000	4,531,000
Very fine sand	912,000	640,000	1,552,000
Totals	16,000,000	16,000,000	32,000,000

TABLE 3

BEDLOAD - KALINSKE FORMULA
REACH NO.1

Grain Size Classification	25-Year Average S = .00023 (tons/year)	Year No. 1 S = .00023 (tons/year)
Very coarse gravel	0	0
Coarse gravel	80	10
Medium gravel	7,780	1,190
Fine gravel	14,100	4,550
Very fine gravel	24,500	13,600
Very coarse sand	40,800	20,300
Coarse sand	75,600	69,500
Medium sand	66,700	67,600
Fine sand	30,600	35,900
Very fine sand	18,000	20,600
Totals	278,160	242,250

TABLE 4

SUSPENDED BED MATERIAL LOAD

Grain Size Classification	Natural Load Modified Flows (tons/year)	Modified Load 10% High Modified Flows (tons/years)
Coarse sand	170,000	150,000
Medium sand	1,060,000	850,000
Fine sand	3,440,000	2,330,000
Very fine sand	8,530,000	5,780,000
Totals	13,200,000	9,110,000

TABLE 5

BED MATERIAL LOAD - EINSTEIN FORMULA

Grain Size Classification	Reach 1 S = .000230 (tons/year)	Reach 1 S = .000265 (tons/year)	Reach 2 S = .000265 (tons/year)
Very coarse gravel	0	20	160
Coarse gravel	540	9,210	4,170
Medium gravel	9,360	98,200	10,410
Fine gravel	36,330	228,380	84,300
Very fine gravel	118,770	507,660	206,230
Very coarse sand	213,160	724,910	408,570
Coarse sand	128,190	571,000	1,352,960
Medium sand	13,620	124,180	1,866,290
Find sand	7,850	78,210	2,530,040
Very fine sand	1,070	1,670	5,600
Totals	650,770	2,343,440	6,468,730

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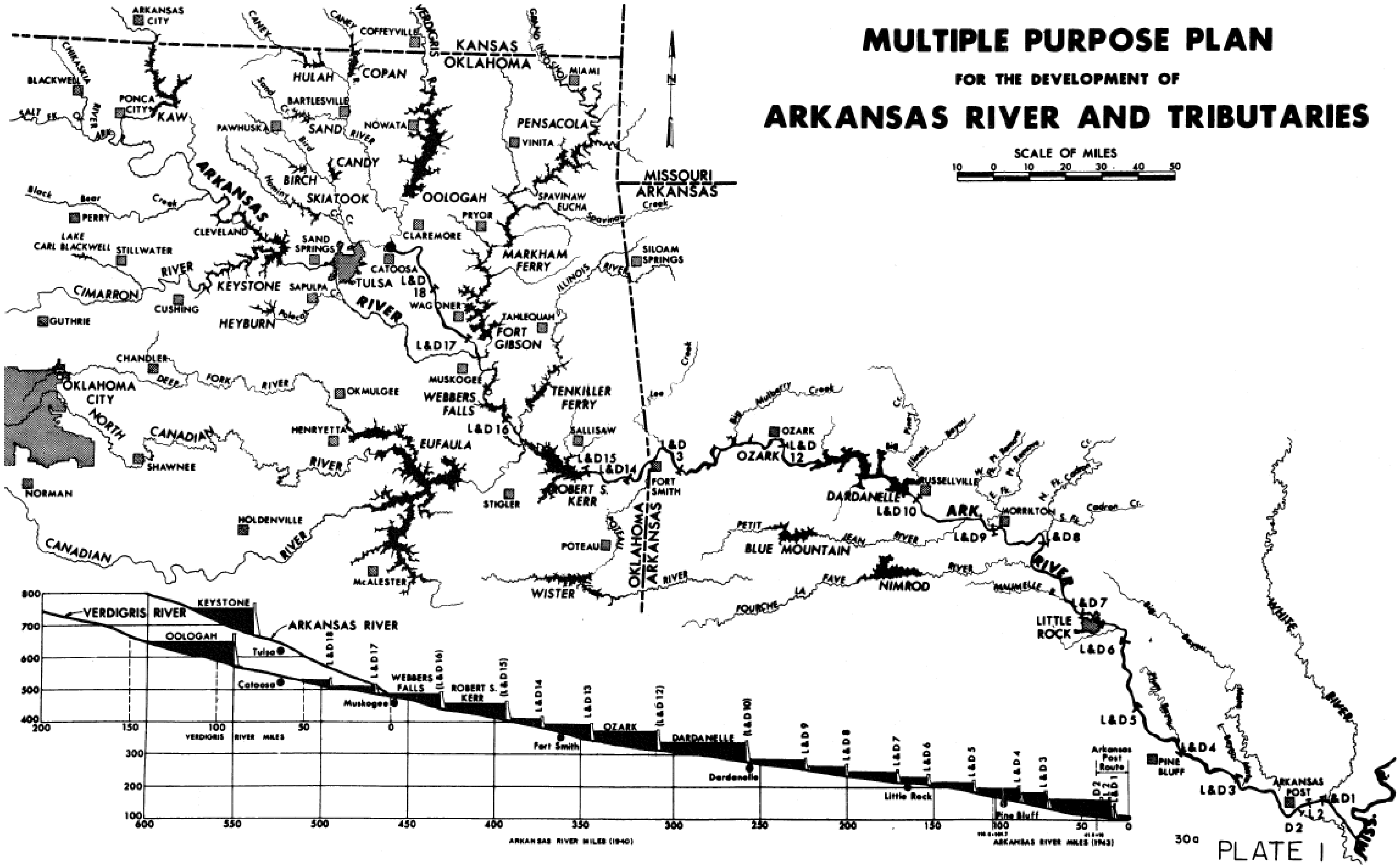
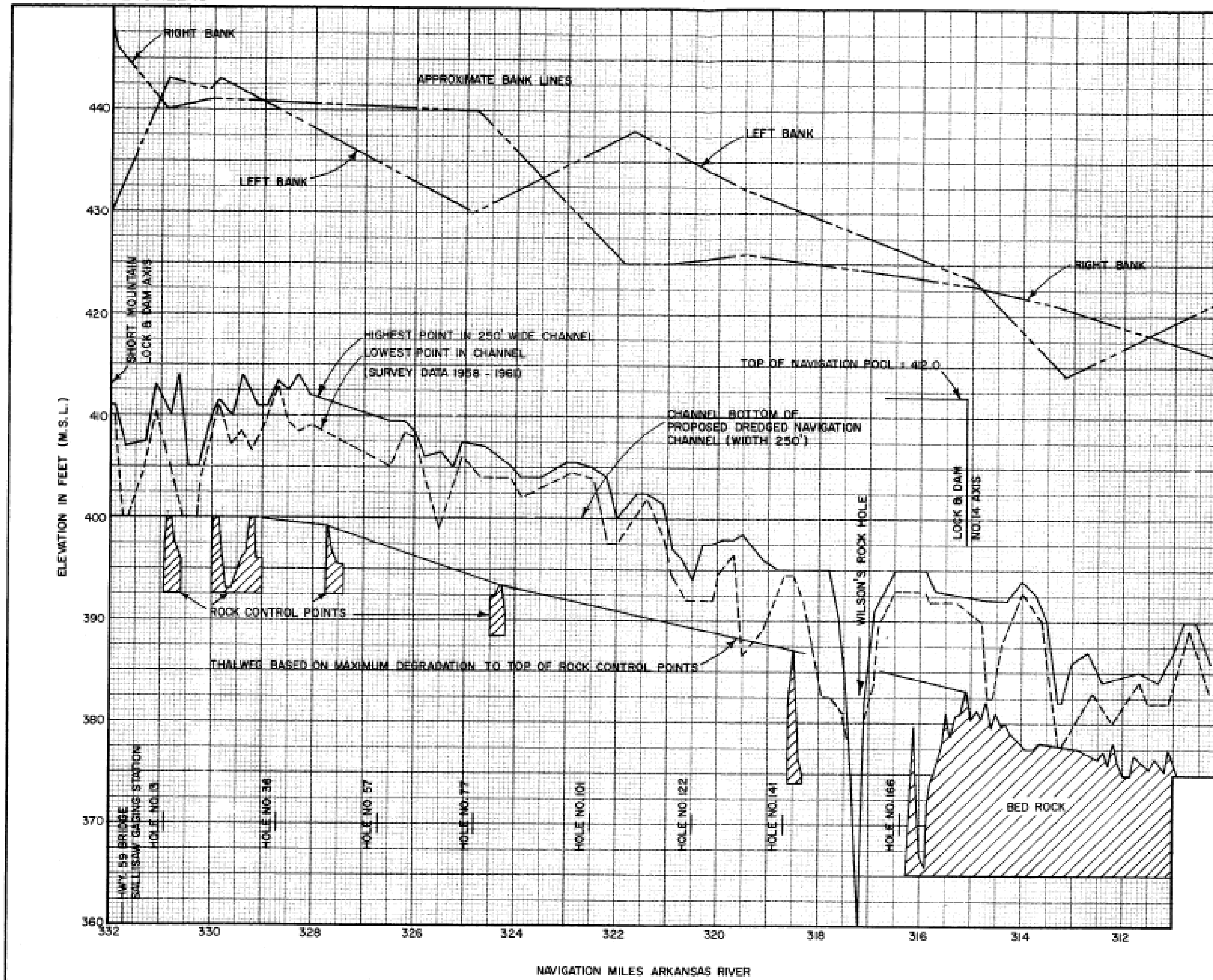
MULTIPLE PURPOSE PLAN
FOR THE DEVELOPMENT OF
ARKANSAS RIVER AND TRIBUTARIES



30a PLATE I

CORPS OF ENGINEERS

U.S. ARMY



ARKANSAS RIVER WATERSHED ARKANSAS RIVER, OKLAHOMA

LOCK AND DAM NO. 15

SHORT MOUNTAIN

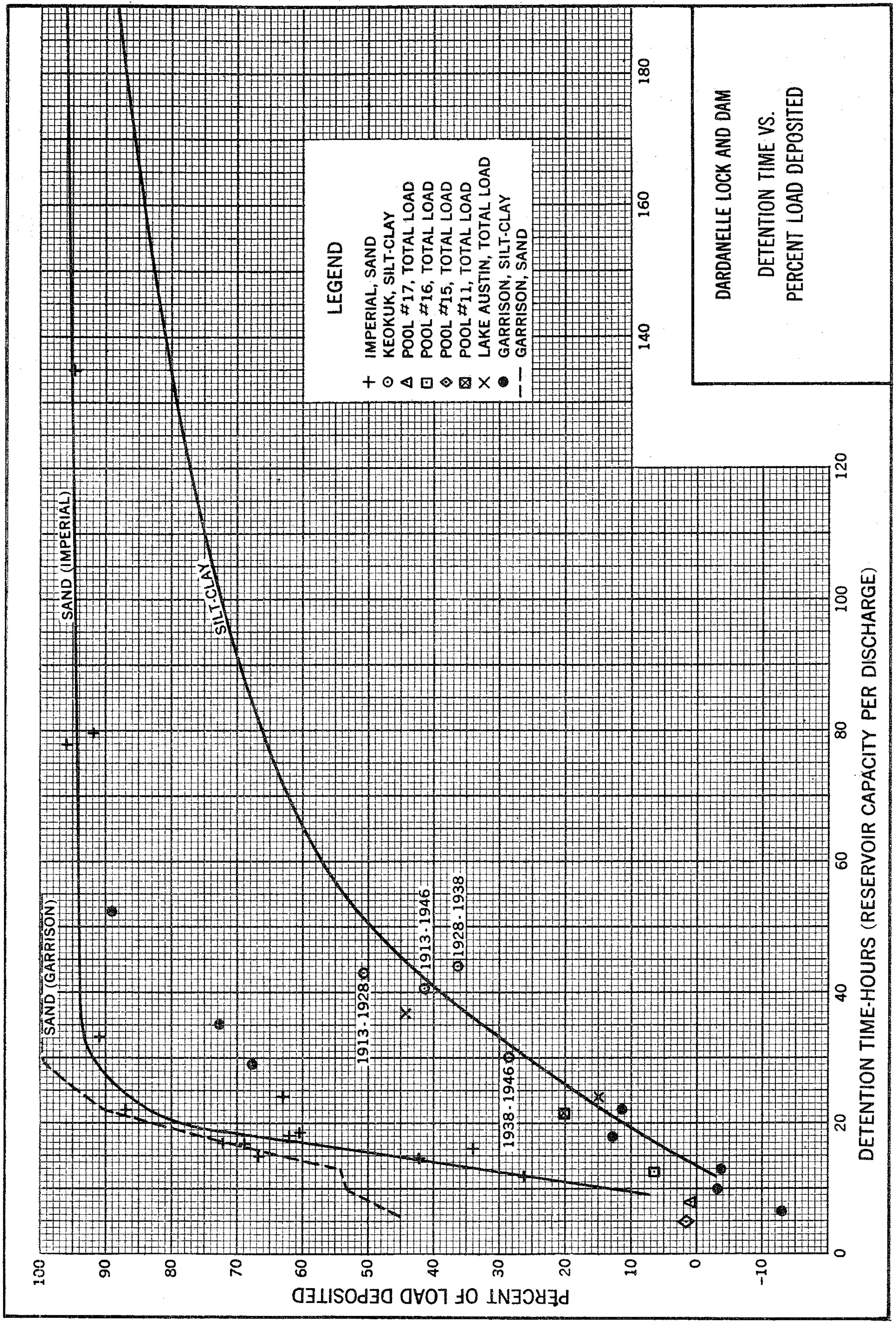
PROFILE OF ARKANSAS RIVER FROM SHORT MOUNTAIN DAM TO LOCK AND DAM NO. 14

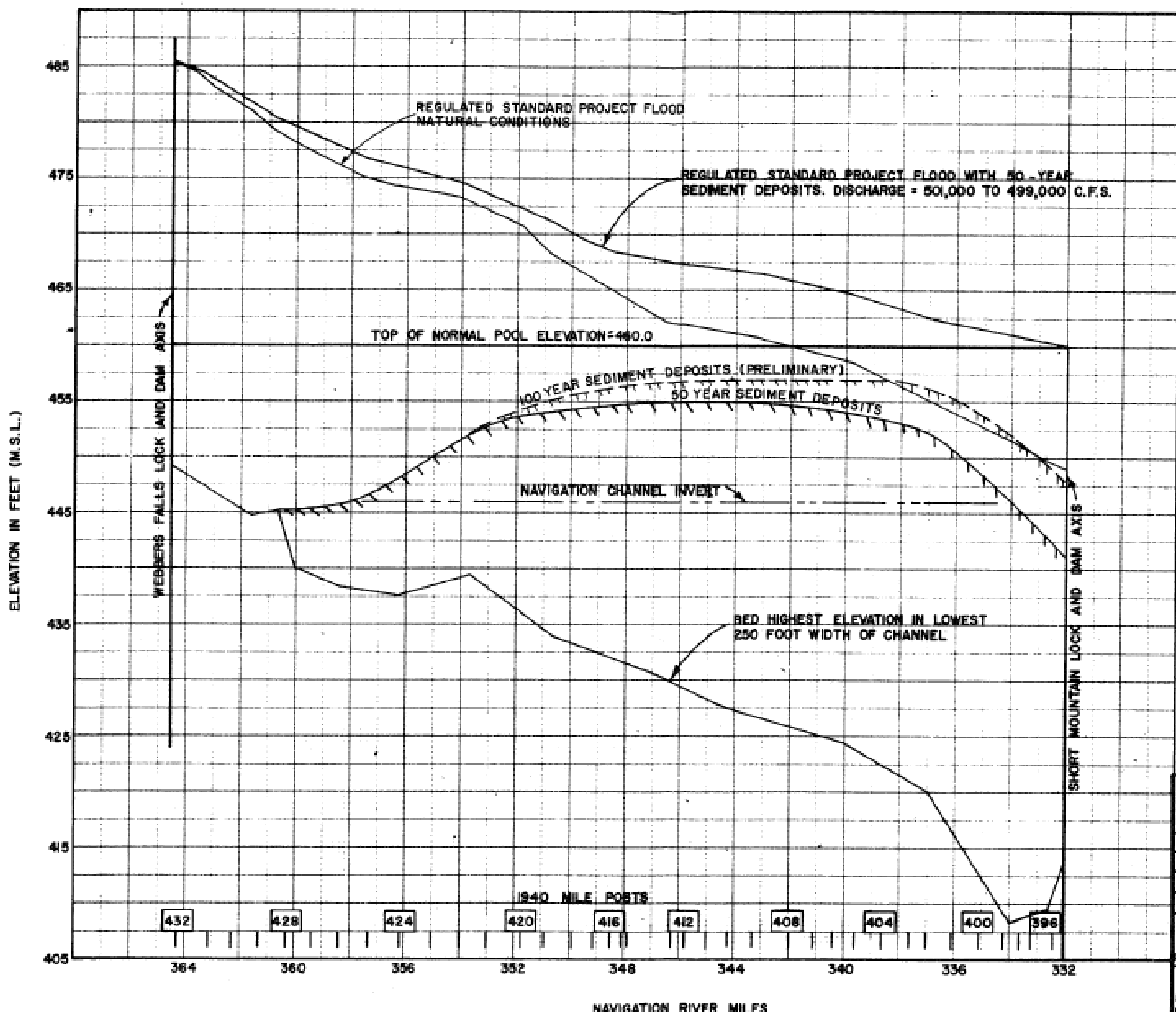
U.S. ARMY ENGINEER DIST., TULSA, CORPS OF ENGINEERS JULY 62

DRAWN: H.O.R.

CHECKED: R.B.C.

PLATE 2


DARDANELLE LOCK AND DAM
DETENTION TIME VS. PERCENT LOAD DEPOSITED
LEGEND
IMPERIAL, SAND
KEOKUK, SILT-CLAY
POOL #17, TOTAL LOAD
POOL #16, TOTAL LOAD
POOL #15, TOTAL LOAD
POOL #11, TOTAL LOAD
LAKE AUSTIN, TOTAL LOAD
GARRISON, SILT-CLAY
GARRISON, SAND
SAND (IMPERIAL)
SAND (GARRISON)
SILT-CLAY
1913-1928
1913-1946
1928-1938
1938-1946
DETENTION TIME-HOURS (RESERVOIR CAPACITY PER DISCHARGE)
PERCENT OF LOAD DEPOSITED



	BY	CHKD	
DESIGNED:	HDR	RBC	
DRAWN:	HDR	RBC	
TRACED:			

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

ARKANSAS RIVER WATERSHED ARKANSAS RIVER, OKLAHOMA

LOCK AND DAM NO. 15
SHORT MOUNTAIN

STANDARD PROJECT
FLOOD PROFILE

SUBMITTED: R. B. Cox
CHIEF, RES CONSV SECT

RECOMMENDED: [signature]
CHIEF, HYD. BRANCH

APPROVED: M. M. Paur
CHIEF, ENGRG. DIVISION

DATE: NOV. 1962

G. W. Morris
COL. CORPS OF ENGINEERS
DISTRICT ENGINEER

SCALE: AS SHOWN

TO ACCOMPANY DESIGN MEMO NO. 3

DRAWING NO. 187-DM3-99/1

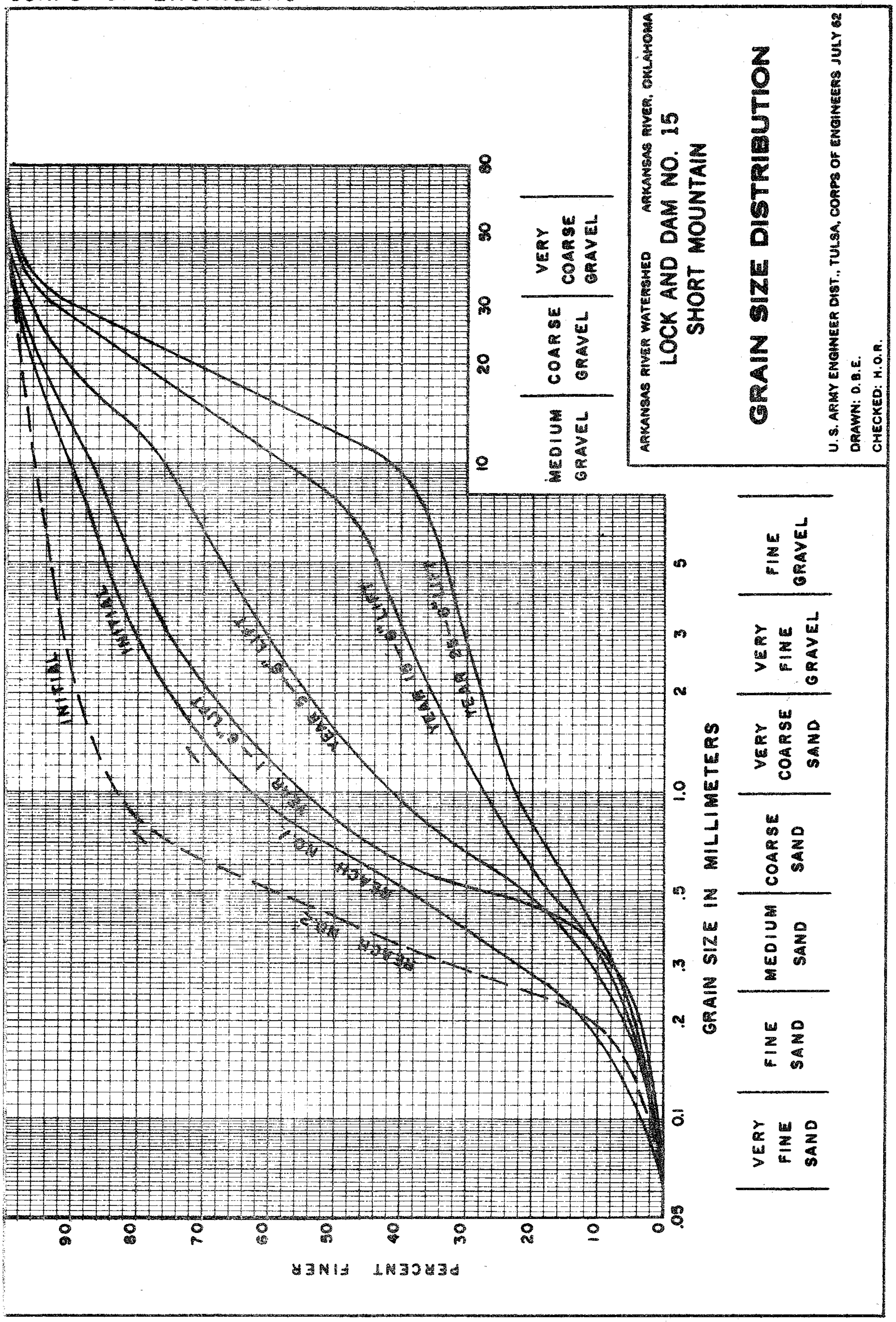
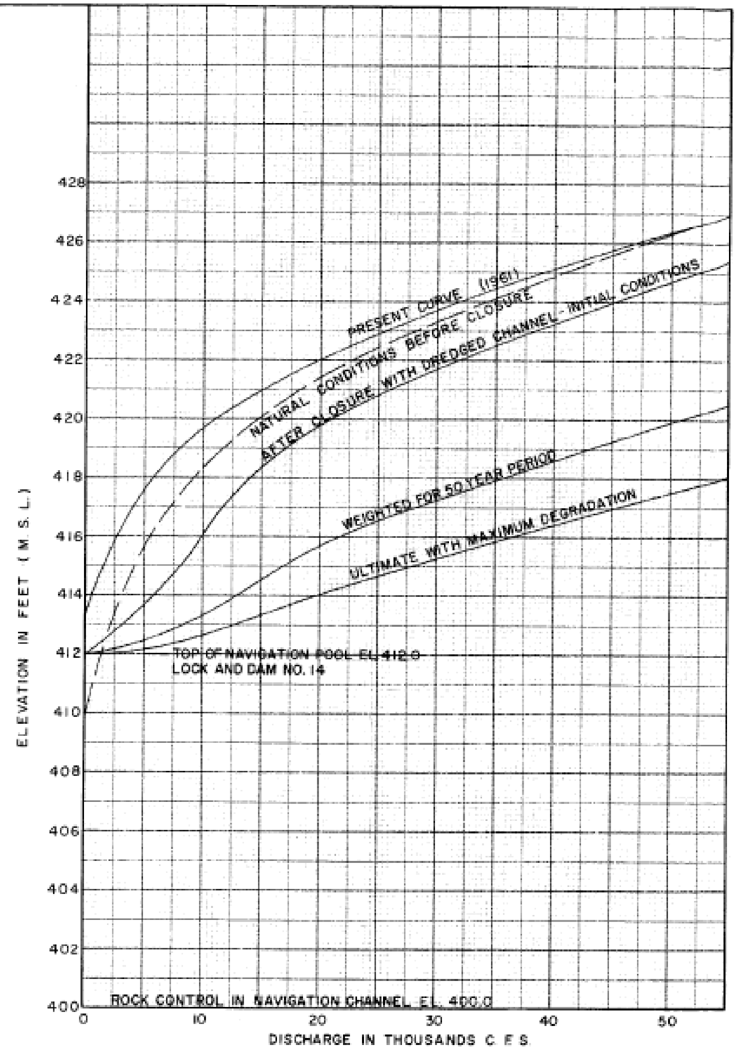
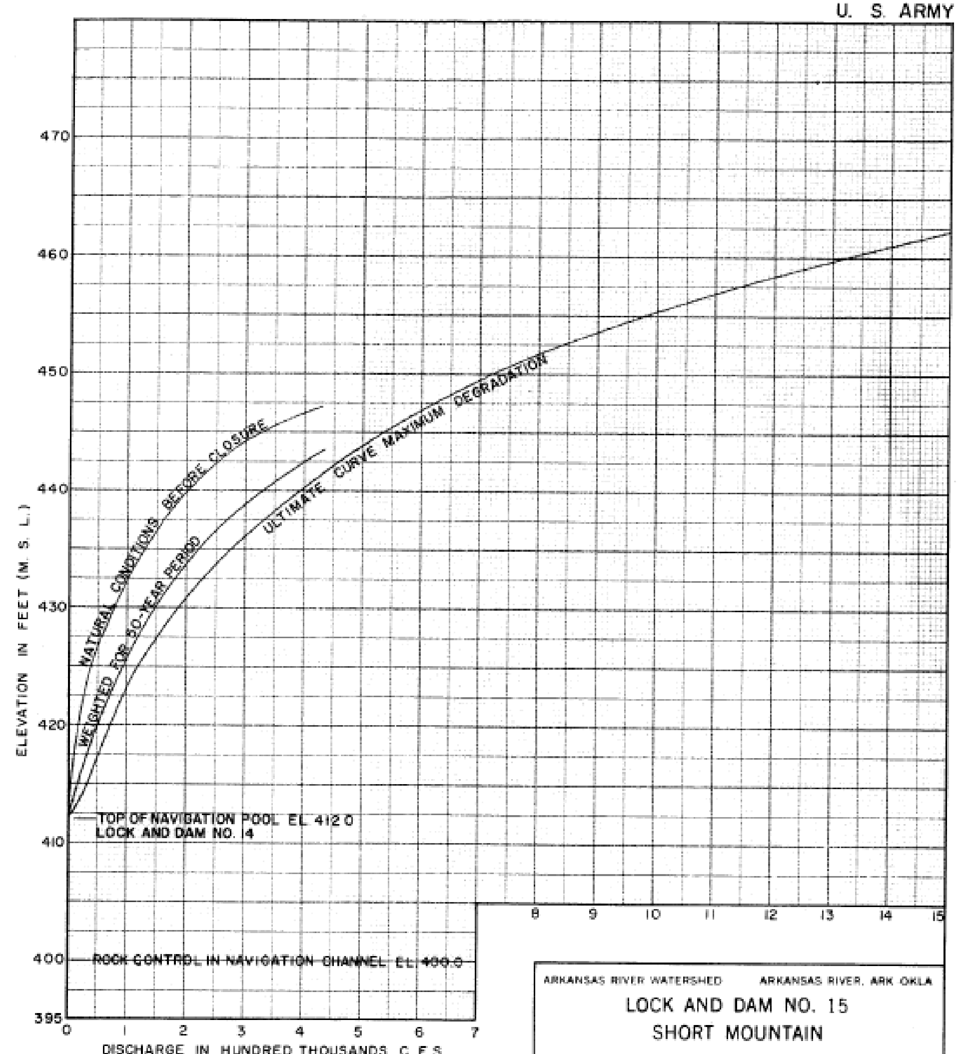

ARKANSAS RIVER WATERSHED ARKANSAS RIVER, OKLAHOMA
LOCK AND DAM NO. 15
SHORT MOUNTAIN
GRAIN SIZE DISTRIBUTION
U.S. ARMY ENGINEER DIST., TULSA, CORPS OF ENGINEERS JULY 62
DRAWN: D.B.E.
CHECKED: H.O.R.
PERCENT FINER
GRAIN SIZE IN MILLIMETERS
VERY FINE SAND
FINE SAND
MEDIUM SAND
COARSE SAND
VERY COARSE SAND
VERY FINE GRAVEL
FINE GRAVEL
MEDIUM GRAVEL
COARSE GRAVEL
VERY COARSE GRAVEL
INITIAL
INITIAL
YEAR 1 – 6" LIFT
YEAR 5 – 6" LIFT
YEAR 15 – 6" LIFT
YEAR 25 – 6" LIFT
REACH NO. 1
REACH NO. 2
.05
0.1
.2
.3
.5
1.0
2
3
5
10
20
30
50
80
0
10
20
30
40
50
60
70
80
90

PLATE 5

CORPS OF ENGINEERS

U. S. ARMY





ARKANSAS RIVER WATERSHED ARKANSAS RIVER, ARK OKLA

LOCK AND DAM NO. 15

SHORT MOUNTAIN

TAILWATER RATING CURVES

U. S. ARMY ENGINEER DIST. TULSA, CORPS OF ENGINEERS JULY 62

DRAWN H. O. R.

CHECKED R. B. C.

A DIGITAL MODEL FOR SIMULATING SEDIMENT MOVEMENT IN A SHALLOW RESERVOIR

by

William A. Thomas[1]

INTRODUCTION

The Ozark Reservoir, plate 1, is on the Arkansas River near Ozark, Arkansas. It is about 36 miles long and 45 feet deep at Ozark Dam. During periods of low flow this run-of-river reservoir, which supplies hydroelectric power and depth for navigation, will be contained within the river channel. However, as in the past, the river will overflow the channel during floods. All gates at Ozark Dam will be fully open when the discharge reaches 475,000 cfs and, except for backwater caused by 2 or 3 feet of swellhead at the dam, essentially open river conditions will exist throughout the reservoir. The recurrence interval for this discharge is about 45 years.

STATEMENT OF THE PROBLEM

The total annual sediment load at the Ozark Reservoir site is estimated to be about 16,000,000 tons/year. In order to determine real estate requirements for the reservoir it is necessary to know the volume of sediment deposits that will accumulate during the life of the project, the location of these deposits and the effect on water surface profiles in the reservoir.

The detention-time method was used to estimate the total volume of deposits; however, this method does not predict the location of deposits or possible scour during flood flows.

Equilibrium bed methods were used to determine the depth of flow for which no deposition or scour would occur under given steady flow discharges and inflowing sediment loads. The sequence of flows is not considered in these methods and hence the rate at which the bed profile will shift to a new equilibrium position is not determined.

In order to account for location of deposition, scour of previously deposited sediments and rate of change of the bed profile, a time-sequence method was developed to relate the hydraulic characteristics of flow, the sediment transport capacity and the conservation of material in successive reaches of the reservoir. The resulting digital model is described in this paper.

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[1]Hydraulic Engineer, The Hydrologic Engineering Center

COMPUTATION PROCEDURE

The name, time-sequence, comes from the fact that to investigate the response of the channel bed to a changing discharge hydrograph it is necessary to study a sequence of flows in the order of their occurrence in the time domain. The independent variables are discharge, duration of discharge, inflowing sediment load, shape and initial bottom elevations of channel cross sections, bottom of dredged channel, a functional relationship between volume of deposits and cross sectional end area, reservoir elevation at the dam and Manning's n-values. The dependent variables are water surface elevation, volume of sediment deposited, bed elevation, and composition of bed material expressed as a percent by weight for each grain size class.

The order of computations is backwater, sediment transport capacity, volume of sediment in a reach and resulting bed elevation. For this study the sequence of discharges should extend in time for a period of 50 years. Such an analytical approach is feasible only by using the electronic computer. This method of simulation is termed digital modeling.

Backwater calculations followed essentially Method II in reference 1. Cross sections, plates 2, 3 and 4, were described by coordinate points and both energy head and Coriolis coefficient were included. From one to ten water surface profiles were determined each computational cycle using the backwater program, and the water surface elevation, effective depth and effective width were output for use in the sediment transport analysis. The significance of effective depths and widths is discussed subsequently in relation to sediment transport capacity.

Sediment transport capacities were calculated using Laursen's relationship[3] as modified by others to fit Arkansas River data[4]. Because of its importance to this problem pertinent parameters are presented here. All symbols are defined in appendix I.

The basic relationship for sediment load is shown on plate 5. This relationship was made more useful by applying the following expressions and plotting sediment load (in tons/day/foot of width) versus the depth-slope product, plate 6:

$$\tau_o = \gamma DS = 28.25 \frac{n^2 V^2}{D^{1/3}} \tag{1}$$

$$\tau_o' = \frac{V^2}{30} \left(\frac{d_m}{D}\right)^{1/3} \tag{2}$$

$$\tau_c = 4d \tag{3}$$

$$q_s = 27\ qc \tag{4}$$

The influence of Manning's n-value on the transport capacity is shown in the insert on plate 6.

The sediment transport capacity, determined from plate 6, is the load in tons/day/foot of width for each grain size class assuming only one grain size is present. To account for a mixture of grain sizes, the sediment transport capacity for each grain size is present in the bed. The total bed material load is calculated by summing the weighted transport capacities for each grain size class.

Also, a procedure was developed by others to relate the sediment ransport capacity to effective width and effective depth rather than top width and hydraulic depth. The effective values are obtained as follows:

$$D_{eff} = \frac{DAD^{2/3}}{\Sigma AD^{2/3}} , \tag{5}$$

$$W_{eff} = \frac{\Sigma AD^{2/3}}{D_{eff}^{5/3}} . \tag{6}$$

Widths and depths are expressed as effective values to account for non-uniform distribution of discharge across the channel. Once the sediment transport capacity is known at the cross sections, the change in sediment load between cross sections can be calculated by:

$$DG = G_{i+1} - G_i \tag{7}$$

where G is the sediment load and the i^{th} cross section is at the downstream end of the reach. The change in sediment load, DG, is added algebraically to previous deposits in the reach, and this volume is expressed in terms of the average end areas at each end of the reach. The depth of deposits are computed by dividing the average end area by the width of cross section over which deposits are expected to accumulate.

This model assumes no scour or deposition at the most upstream cross section thereby making it possible to calculate the depth at the downstream end of this reach. In succeeding reaches the depth of deposits at the upstream end is the same as that calculated for the downstream end of the previous reach, and the resulting bed profile is continuous through the reservoir.

The cross section elevations, over that portion where deposits are expected, are changed by the calculated sediment depth. Navigation depth is checked and dredging performed if required. This completes a cycle, and the next sequence of discharges are entered to advance the calculations along the time axis.

DESCRIPTION OF DIGITAL MODEL

The digital model is made up of the geometric model, hydrologic model, inflowing sediment load and the composition of the bed material.

Geometric model - The geometric model is described by eight cross sections as shown on plate 1 and is termed the 8-section model. Average sections developed for this model are shown on plates 2, 3, and 4. These were formed from 40 cross sections of the reservoir using a 2-step procedure.

First, Manning's n-values were calculated for several discharges using measured water surface elevations and the 40 cross sections. Second, locations for the eight average sections were determined, and the 40 cross sections were divided into eight groups.

The average sections were developed from the thalweg elevations, the elevations of the lowest 300 foot width, the elevations and widths at the construction reference plane (water surface profile for approximately 10,000 to 15,000 cfs discharge), the elevations and widths at top bank and the pertinent elevations and widths in the overbanks of all sections in each group.

Backwater profiles were calculated using the 8-section model and Manning's n-values determined in the first step. The average sections were adjusted slightly until the water surface profiles for the 8-section model matched those of step one. The geometric model was then considered to be verified to preproject conditions.

Hydrologic model - The sequence of flows selected for the hydrologic model was mean daily discharges, modified by all authorized upstream projects, for the period 1923 through 1961. This 39-year period of record was extended to 50 years by reusing the first 11 years of record.

The discharge hydrograph was changed to a discharge histogram for use in the steady-flow, backwater program. Discharges between 25,000 and 175,000 cfs were grouped in time to obtain a sequence of flows in increments of 50,000 cfs (i.e., 50,000, 100,000 or 150,000 cfs). The time duration of these groups ranged from 88 to 2 days. Discharges less than 25,000 cfs were disregarded because the sediment load is small, and above 175,000 cfs the flows were grouped into durations of 1 to 3 days using whatever value of discharge that best described the flow. The histogram was balanced so that each average period contained the same discharge volume as the original hydrograph. This procedure is comparable to that used in movable-bed, hydraulic model studies.

Sediment load - The inflowing sediment load was calculated previously by others and presented in table 35 of reference 5. The inflowing bed material load for the first five years was averaged, and the load for the remaining 40 years was averaged to provide two rates of sediment inflow during the life of the project as shown in the following table.

Inflowing Bed Material Load

Bed Material Sand Load in 1000 Tons/Day

Years	Coarse	Medium	Fine	Very Fine	Q 1000 cfs
1-5	.19	.49	.80	.19	7.6
	.54	2.30	6.60	1.70	30.0
	3.00	15.00	43.00	11.80	100.0
	90.00	265.00	430.00	120.00	430.0
6-50	.47	.57	.14	.14	7.6
	1.55	2.25	1.23	1.23	30.0
	4.10	15.00	9.00	8.50	100.0
	195.00	500.00	175.00	150.00	600.0

These inflowing rates reflect a reduction in the total sediment load resulting from the authorized upstream projects.

Two cases were studied. In case I the inflowing sediment load was separated into four grain size classes: coarse, medium, fine and very fine sand. Particle sizes for each class are shown on plate 6. The second case utilized only the median grain size, medium sand, and was studied because the equilibrium bed methods considered only this grain size. Accordingly, the results of the time-sequence method could be compared to those from the previous, equilibrium bed studies.

Bed material - The grain size distribution in the channel bed was used in determining the contribution of each grain size class to the total sediment transport capacity. Values developed in previous studies, reach v, table 5 of reference 2, were accepted for initial conditions. These are shown in the following table.

Bed Composition

Grain Size Fraction	Amount Present in Bed in Percent
Coarse	19
Medium	37
Fine	39
Very Fine	5

The grain size distribution in the bed changed as sediment was being deposited. These changes were determined after each discharge was analyzed.

All deposited material was permitted to scour during subsequent flows. However, the initial channel bed in the reservoir was permitted to scour only 2.7 feet deep. This is the calculated depth required to armour the bed.

VERIFICATION OF THE MOVABLE BED MODEL

Before making the final calculations, it was necessary to verify that the 8-section model would describe the sediment motion in the river. Information relating water discharge, sediment load and channel geometry was limited to the survey from which the 8-section model was developed. Therefore, using that discharge, 50,000 cfs, the corresponding sediment load, natural river conditions and the 8-section model, 1200 days of steady flow were simulated. A total sand load of 7 acre-feet passed through the model and changes to the bed elevation were negligible. This indicates the 8-section model would describe the sediment conditions from which it was formed, and the model was considered to be verified.

RESULTS OF THE STUDIES

Fifteen hours of computer time were required to simulate one 50-year period flow histogram*. Printed output was obtained showing each set of backwater profiles and the resulting bed profiles. Plate 7 shows the bed profiles for initial conditions, when the pool is first formed, and after 10, 25 and 50 years of operation. Deposits were distributed in the cross sections as shown on plates 2, 3 and 4. The change in depth of deposits as a function of time is shown on plate 8 for a sample period of record. This plate illustrates that during floods, the bed elevation does not degrade instantaneously at all points along the channel, but that the sediment material moves reach by reach as fast as the law of continuity permits.

COMPARISON WITH OTHER METHODS

Results from the detention-time method and the equilibrium bed method are shown on plate 9. The case II bed profiles on this plate were developed for comparisons with the equilibrium bed method. The following table shows volumes obtained by the three methods.

Total Volume of Sediment Deposits

Computation Procedure	Time in Years	Volume in 1000 ac.ft.
Case I	50	52.5
Case II	50	37.5
Detention-time	50	45.8
Equilibrium bed (1)		36.6

(1) Laursen's transport relationship

* A GE 225 digital computer with 8k memory was used.

The close agreement between case II and the equilibrium bed method suggests the sediment transport model did obey the law of continuity.

CONCLUSIONS

The state of the art is such that a general analysis of sediment deposition can be simulated in more detail using the electronic computer and digital models than is possible with the detention-time or equilibrium bed methods. The results of the Ozark study show reasonable agreement among the total volumes determined by the three methods, and the time-sequence method provides not only total volume but also rate and location of deposition and scour for a number of different grain size classes.

The problems involved with digital models are similar to those involving movable-bed physical models. The model geometry and roughness coefficients have to be determined, the water discharge hydrograph translated into a suitable histogram and the model verified to experienced conditions.

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3. Laursen, Emmett M., "The Total Sediment Load of Streams", Journal of the Hydraulics Division, American Society of Civil Engineers, Proceeding Paper 1530, February 1958.

4. "Normal Pool Elevations", Project Design Memorandum No. 5-3, Arkansas and Oklahoma, Little Rock District, Corps of Engineers, May 1960, Appendix I.

5. Supplement to Project Design Memorandum No. 6-4, Sedimentation, Dardanelle Reservoir, Arkansas River and Tributaries, Arkansas and Oklahoma, Little Rock District, Corps of Engineers, 23 January 1959.

APPENDIX I

SYMBOLS

A - Area of cross section in square feet.

c - Concentration of sediment in percent by weight.

D - Depth of flow in feet.

d - Diameter of sediment particle in feet (geometric mean diameter of fractional size range).

d_m - Median size of sediment particles in the stream bed in feet (considered representative of the grain roughness of the bed).

n - Manning's n-value.

q - Unit water discharge in cfs/ft width.

q_s - Unit sediment load in tons/day/foot width.

S - Slope of energy gradient.

T - Top width of cross section at water surface.

V - Average velocity of flow = $(Q/A)\sqrt{\alpha}$ in fps.

w - Fall velocity of sediment particle in fps.

γ - Unit weight of water (62.4 lbs/cu.ft.).

τ_o - Boundary shear at stream bed in lbs/sq.ft.

τ_c - Critical boundary shear for beginning of movement of sediment particles.

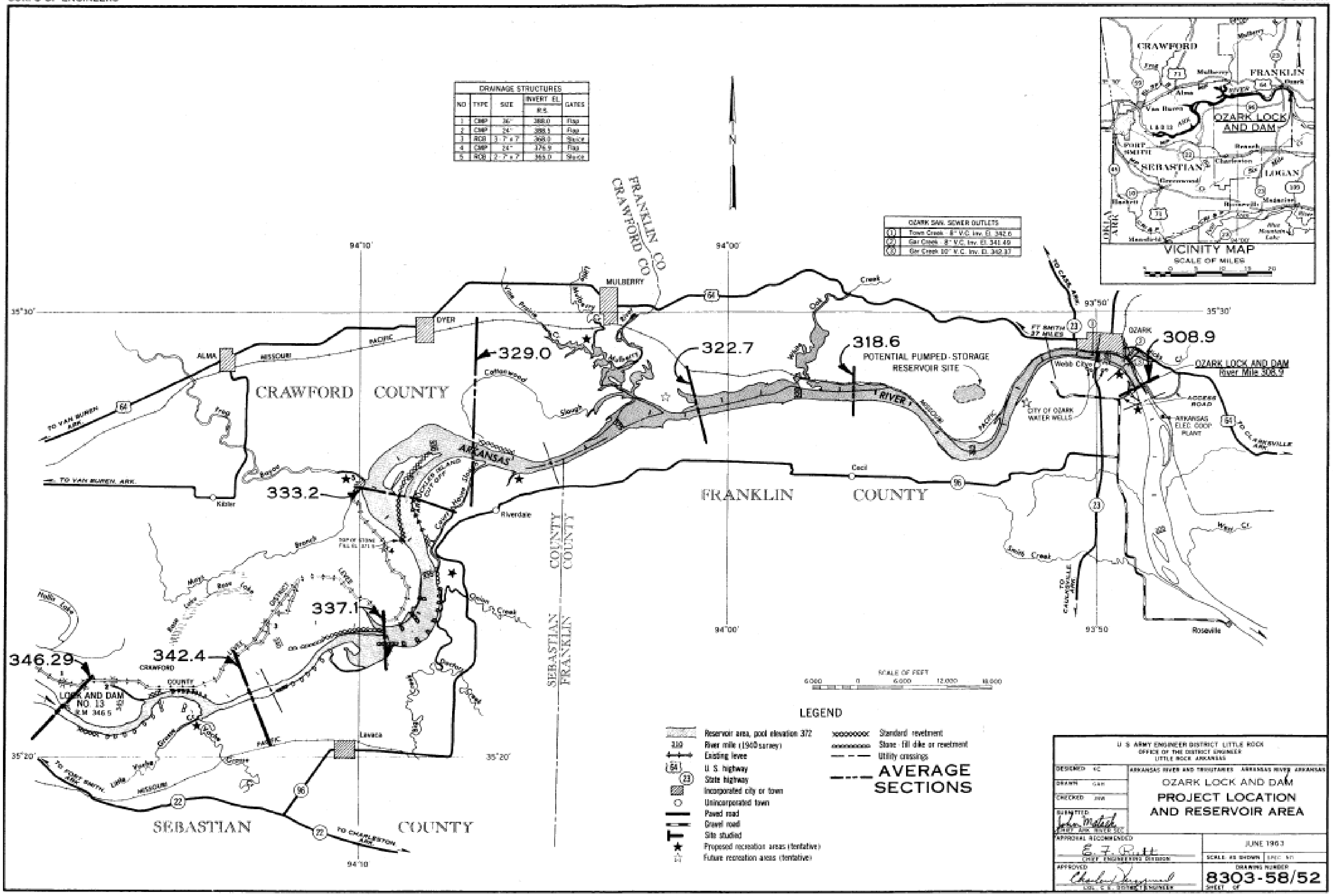
τ_o' - Boundary shear associated with the sediment particles in the stream bed.

ρ - Mass density of fluid (1.94 slugs/ft^3).

ACKNOWLEDGEMENT

Work on this model was accomplished while the author was employed by the Little Rock District, Corps of Engineers in the Hydraulics Branch. The assistance rendered by that branch in providing plates and other data for this paper is gratefully acknowledged.

CORPS OF ENGINEERS | U. S. ARMY



DRAINAGE STRUCTURES				
NO	TYPE	SIZE	INVERT EL R.S.	GATES
1	CMP	36"	388.0	Flap
2	CMP	24"	388.5	Flap
3	RCB	3 - 7' x 7'	368.0	Sluice
4	CMP	24"	376.9	Flap
5	RCB	2 - 7' x 7'	365.0	Sluice

OZARK SAN. SEWER OUTLETS	
(1)	Town Creek - 8" V.C. Inv. El. 342.6
(2)	Gar Creek - 8" V.C. Inv. El. 341.49
(3)	Gar Creek 10" V.C. Inv. El. 342.37

U. S. ARMY ENGINEER DISTRICT LITTLE ROCK
OFFICE OF THE DISTRICT ENGINEER
LITTLE ROCK ARKANSAS

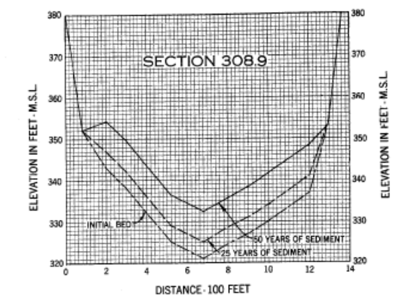
ARKANSAS RIVER AND TRIBUTARIES ARKANSAS RIVER ARKANSAS

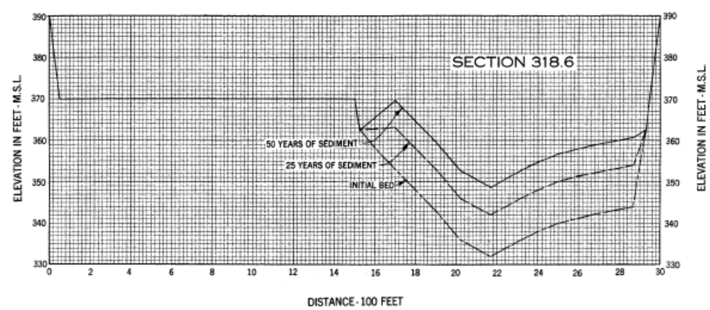
OZARK LOCK AND DAM
PROJECT LOCATION
AND RESERVOIR AREA

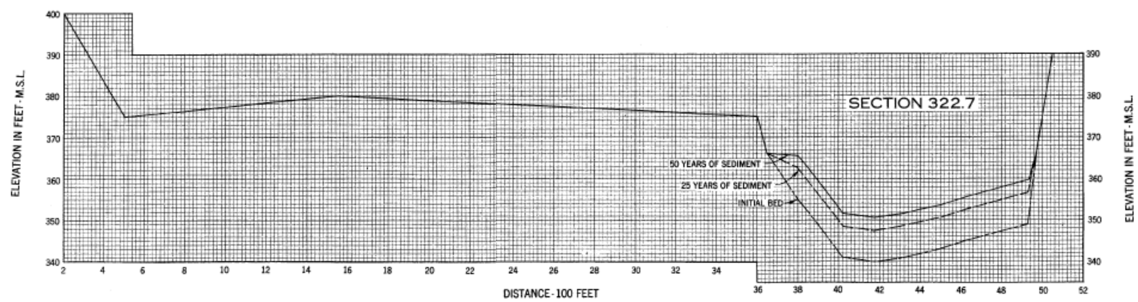
DESIGNED: KC
DRAWN: GAH
CHECKED: JWW
SUBMITTED: John Matash, CHIEF ARK. RIVER SEC.
APPROVAL RECOMMENDED: E. F. Rutt, CHIEF ENGINEERING DIVISION
APPROVED: Charles [illegible], LT. COL. C. E. DISTRICT ENGINEER

JUNE 1963
SCALE: AS SHOWN
DRAWING NUMBER 8303-58/52

PLATE 1







ARKANSAS RIVER WATERSHED — ARKANSAS AND OKLAHOMA

OZARK LOCK AND DAM

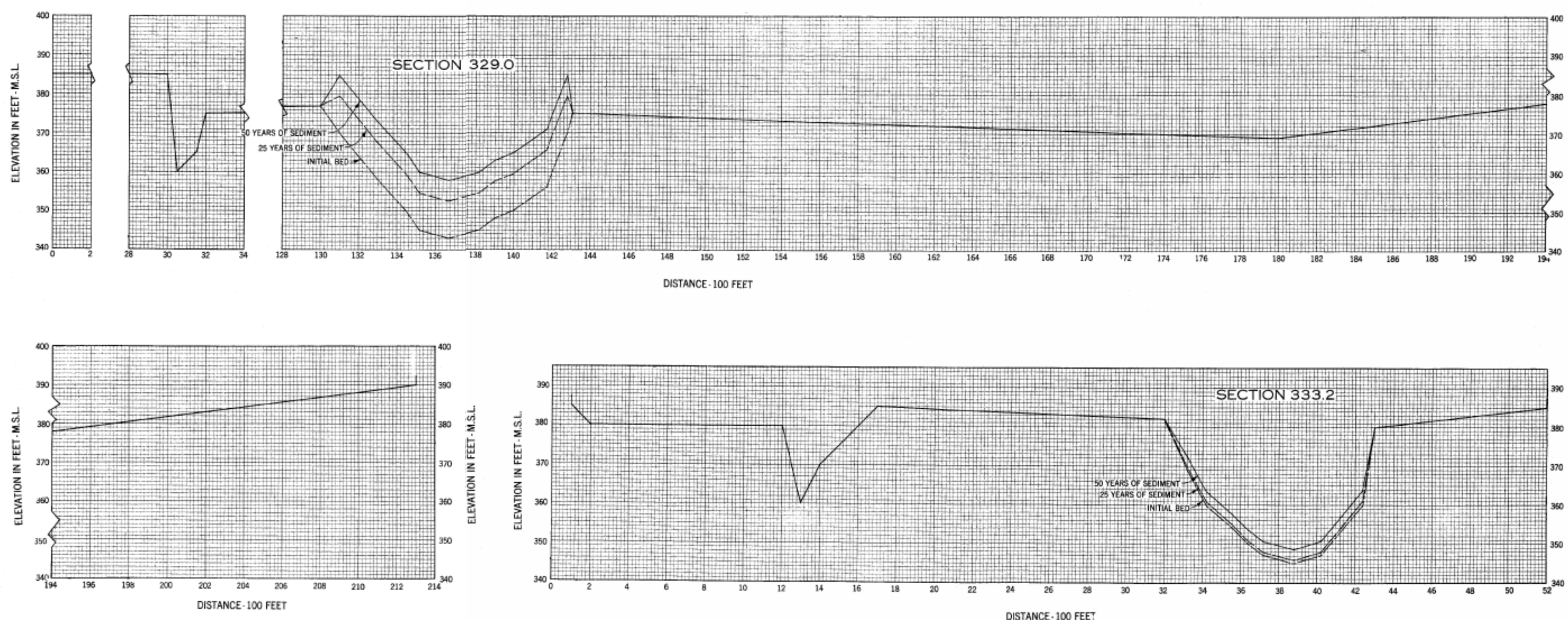
AVERAGE SECTIONS USED FOR TIME SEQUENCE METHOD

SCALE: AS SHOWN

IN 3 SHEETS — SHEET NO 1

U. S. ARMY ENGINEER DISTRICT, LITTLE ROCK
LITTLE ROCK, ARKANSAS JANUARY 1968

PLATE 2



ARKANSAS RIVER WATERSHED — ARKANSAS AND OKLAHOMA

OZARK LOCK AND DAM

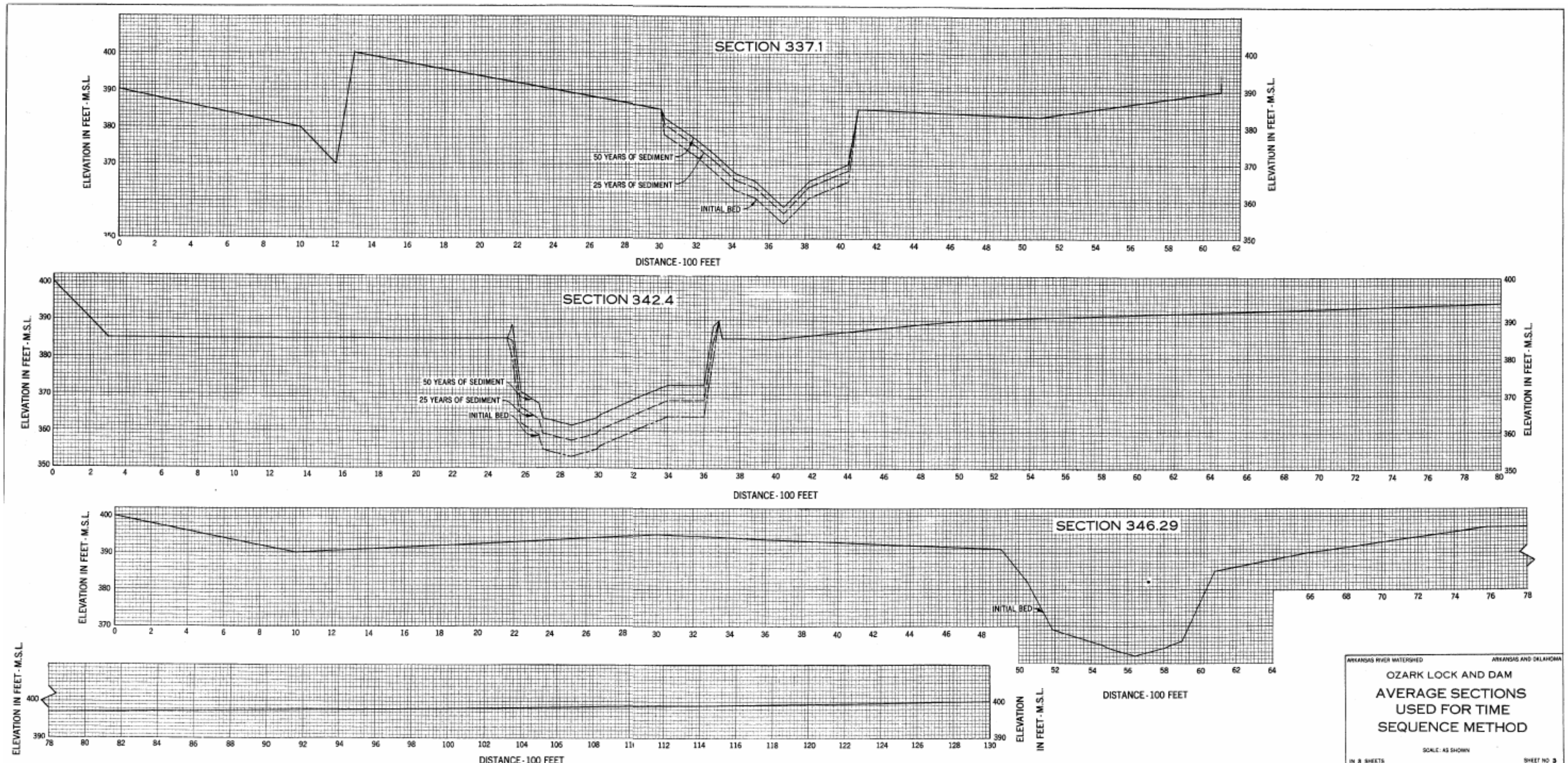
AVERAGE SECTIONS USED FOR TIME SEQUENCE METHOD

SCALE: AS SHOWN

IN 3 SHEETS — SHEET NO. 2

U. S. ARMY ENGINEER DISTRICT, LITTLE ROCK
LITTLE ROCK, ARKANSAS JANUARY 1968

PLATE 3



ARKANSAS RIVER WATERSHED — ARKANSAS AND OKLAHOMA

OZARK LOCK AND DAM

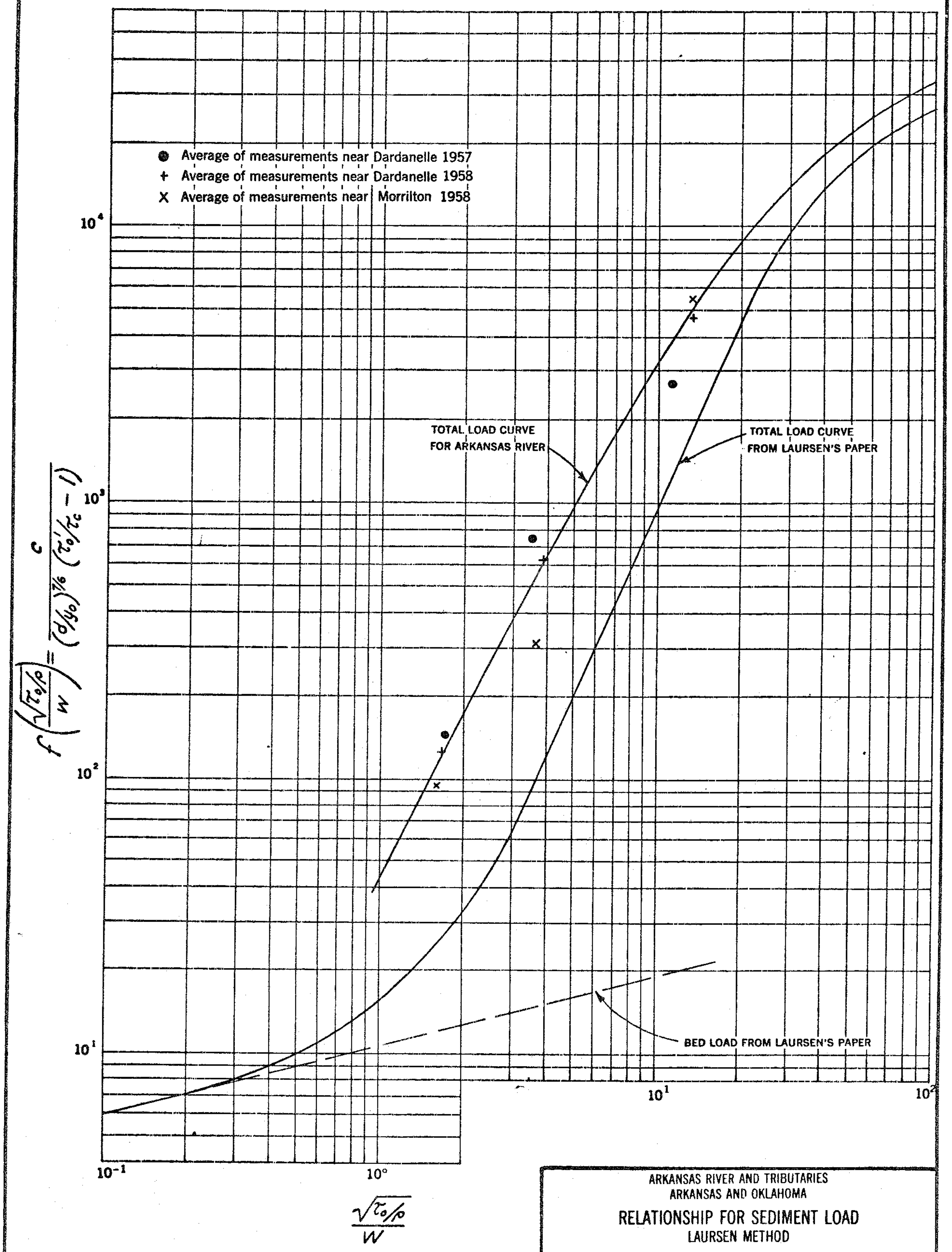
AVERAGE SECTIONS
USED FOR TIME
SEQUENCE METHOD

SCALE: AS SHOWN

IN 8 SHEETS — SHEET NO 3

U. S. ARMY ENGINEER DISTRICT, LITTLE ROCK
LITTLE ROCK, ARKANSAS JANUARY 1968

PLATE 4



ARKANSAS RIVER AND TRIBUTARIES
ARKANSAS AND OKLAHOMA

RELATIONSHIP FOR SEDIMENT LOAD
LAURSEN METHOD

SCALE: AS SHOWN

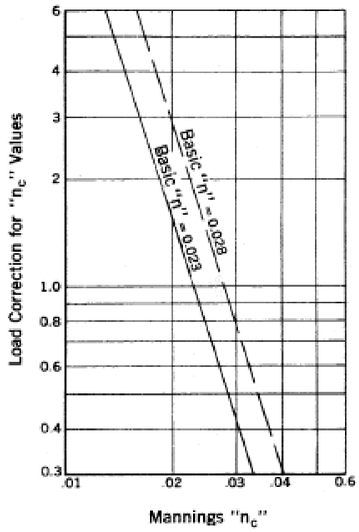
U. S. ARMY ENGINEER DISTRICT, LITTLE ROCK
LITTLE ROCK, ARKANSAS, MAY 1960

DESIGNED: OBY
DRAWN: JDS
CHECKED: JCP

TO ACCOMPANY PROJECT DESIGN
MEMORANDUM NO. 5-3
DATED MAY 1960

SERIAL 7881

115/776

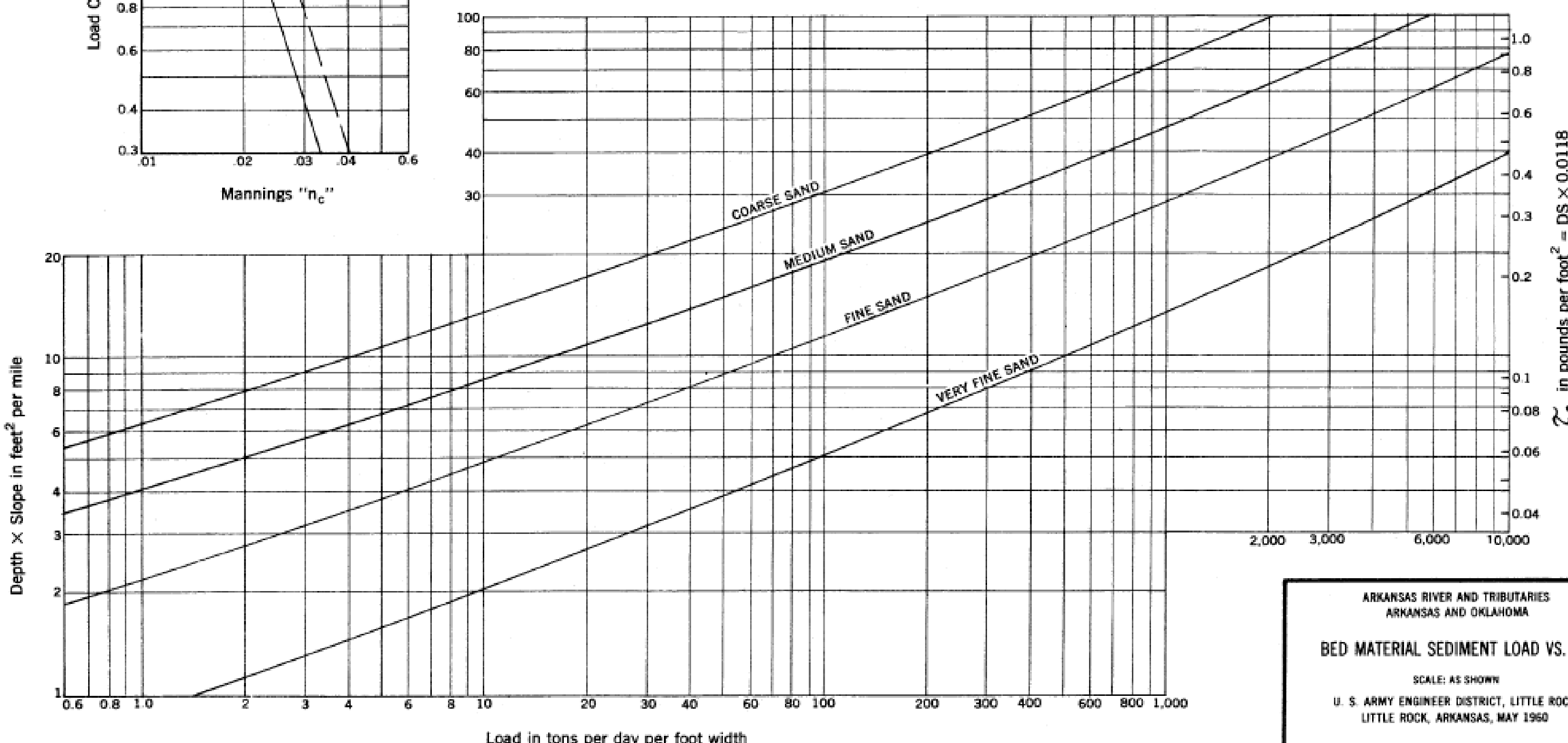


NOTES:

Median size of bed material = 0.25 m.m.
Temperature 65°F

Coarse Sand, 1.0 - 0.5 m.m., geometric mean size 0.707 m.m.
Medium Sand, 0.5 - 0.25 m.m., geometric mean size 0.354 m.m.
Fine Sand, 0.25 - 0.125 m.m. geometric mean size 0.177 m.m.
Very Fine Sand, 0.125 - 0.062 m.m., geometric mean size 0.088 m.m.

Load corrections apply to sediment load values read from sediment load vs. DS. curves



ARKANSAS RIVER AND TRIBUTARIES
ARKANSAS AND OKLAHOMA

BED MATERIAL SEDIMENT LOAD VS. DS.

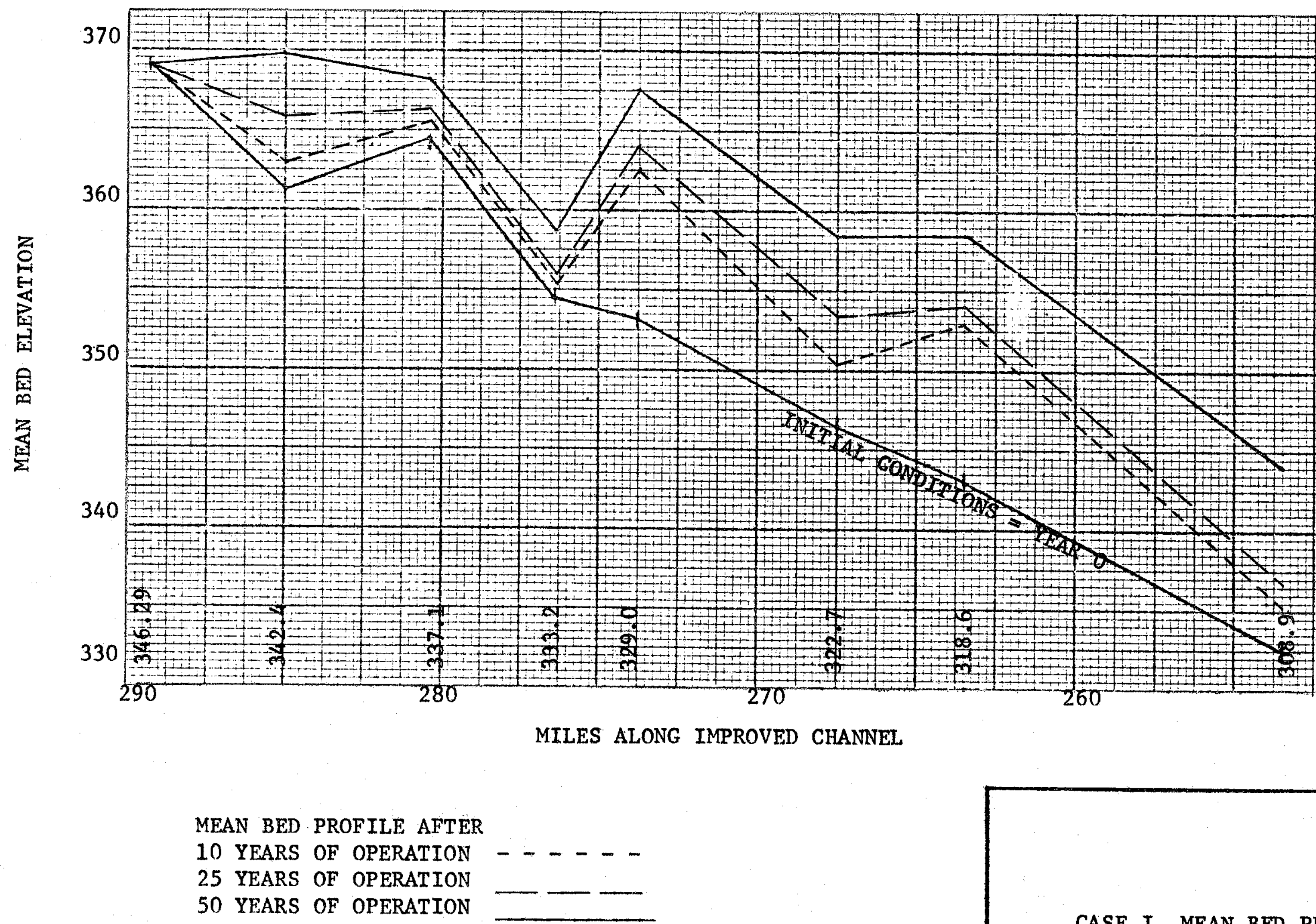
SCALE: AS SHOWN

U. S. ARMY ENGINEER DISTRICT, LITTLE ROCK
LITTLE ROCK, ARKANSAS, MAY 1960

DESIGNED: DBY
DRAWN: JMB
CHECKED: JCP

TO ACCOMPANY PROJECT DESIGN MEMORANDUM NO. 5-3 DATED MAY 1960

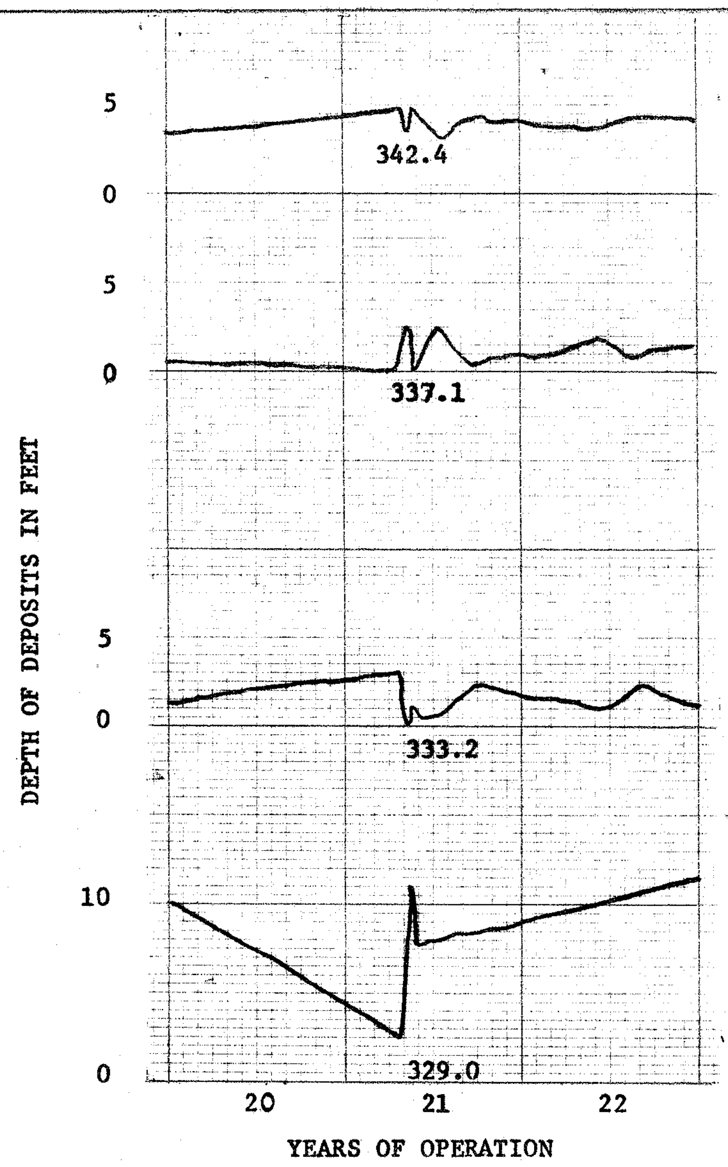
SERIAL 7881
115/777

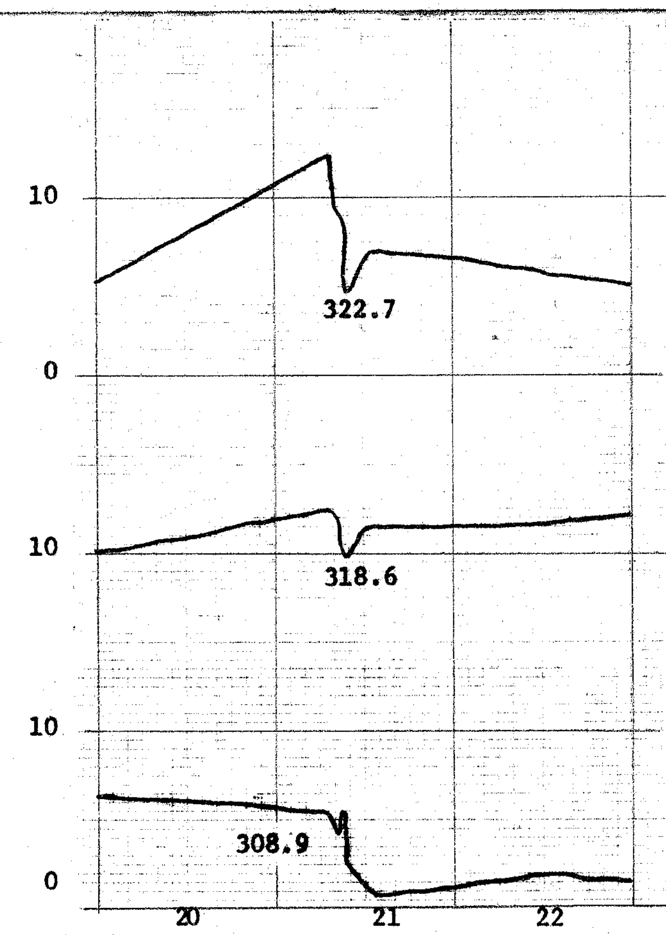


CASE I MEAN BED PROFILES

TIME-SEQUENCE METHOD

OZARK RESERVOIR

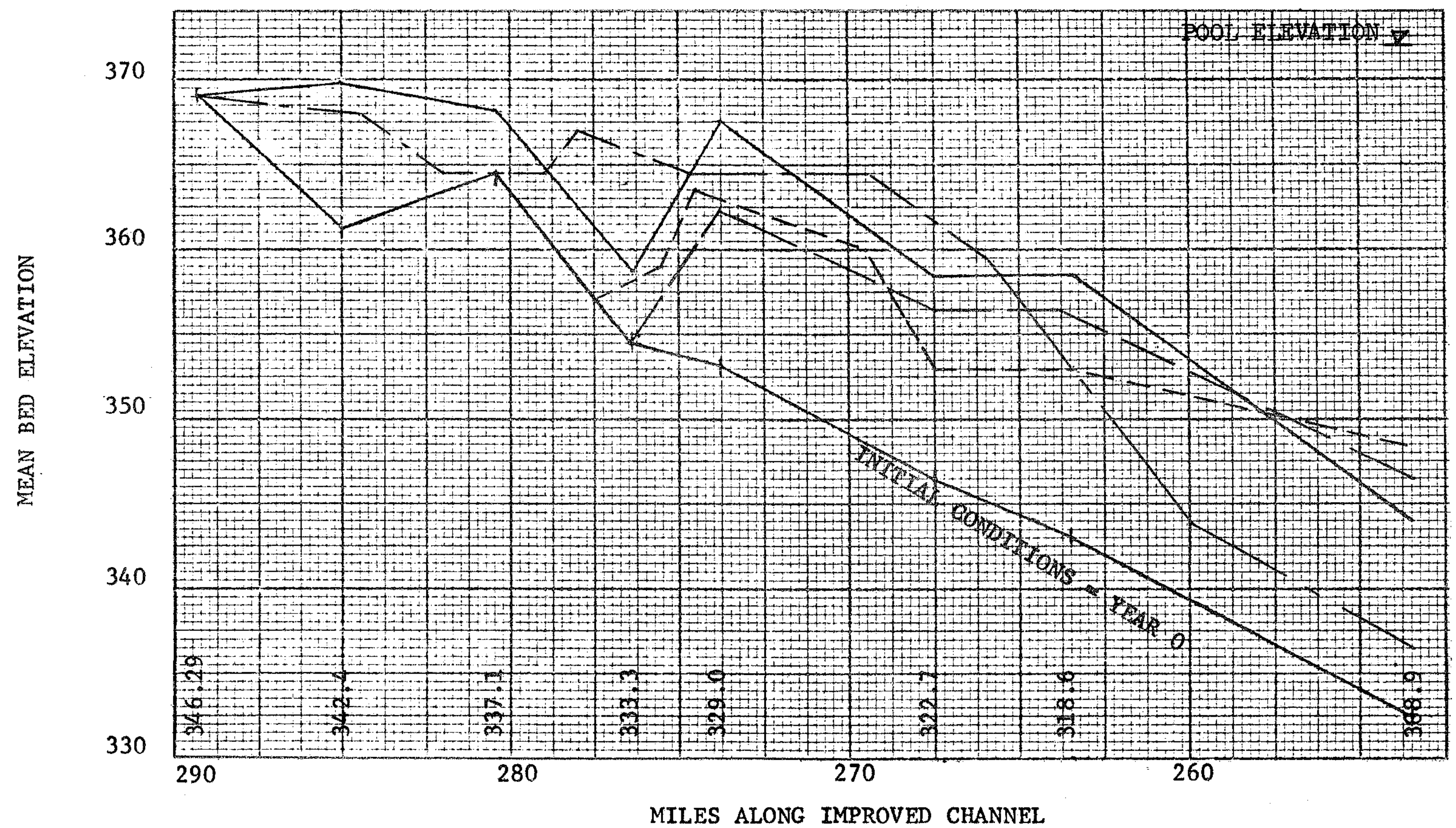




Note: YEAR 21 REFLECTS THE 1943 FLOOD
(MODIFIED MEAN DAILY FLOW = 504,000 CFS)

DEPTH OF DEPOSITS
AT THE AVERAGE SECTIONS
TIME-SEQUENCE METHOD

OZARK RESERVOIR



AFTER 50 YEARS OF OPERATION
DETENTION-TIME
EQUILIBRIUM BED (1)
CASE I
CASE II

(1) NO TIME INTERVAL ASSOCIATED WITH THE EQUILIBRIUM BED METHOD

TIME-SEQUENCE PROFILES COMPARED TO OTHER METHODS

OZARK RESERVOIR

THE KANSAS CITY DISTRICT SUSPENDED SEDIMENT LOAD COMPUTER PROGRAM

by

Charles H. Sullivan[1]

The Kansas City District has operated about 40 suspended sediment stations for the last ten years. The District started its suspended sediment sampling in 1942 with nine stations. The method for making daily load computations is the same today as when our program was started. The method used for making suspended sediment load computation, which I will refer to as hand computations, has given very satisfactory results. With 40 stations to make suspended sediment load computation for each year, a considerable number of man-days is involved in routine computations. It was decided to investigate the possibilities of using the electronic computer in suspended sediment load computations.

As mentioned, the hand computation has given very satisfactory results for 28 years. Therefore, a method for reproducing hand computations was desired. I would like to describe the hand computation method so that the computer program can be compared to it. Figure 1 and Figure 2 represent the first steps in normal hand computation for a suspended sediment station. The data shown are for the months of June and July 1967 at the sampling station on the Little Blue River near Barnes, Kansas. First, daily average discharges, as published by the U.S. Geological Survey, are plotted for the two-month period on semilogmetric graph paper (Fig. 1 and Fig. 2). The results of the analyses of suspended sediment samples are plotted as grams per liter for their proper time period. These are shown on Figure 1 and Figure 2. From these samples a curve is drawn to represent the change in suspended sediment concentrations as related to changes in flow. The mean daily suspended load computation is made as shown in equation 1.

Eq (1) daily load in tons

$$= \text{average daily flow(c.f.s.)} \times \text{average suspended sediment concentration(G/L)} \; \frac{\times\ (24\ \text{hrs/day} \times 60\ \text{min/hr} \times 60\ \text{sec/min} \times 62.4\ \text{lbs/cu. ft. water})}{1{,}000\ \text{g/liter} \times 2{,}000\ \text{lbs/tons}}$$

or

$$= \text{average daily flow(c.f.s.)} \times \text{average suspended sediment concentration(G/L)} \times 2.7$$

[1]Hydraulic Engineer, Kansas City District

The daily average flow, as published by the U.S. Geological Survey, is used in the computations. The daily average suspended sediment concentration is picked from the concentration curve shown in Figure 1 and Figure 2. For days such as 16 July, values can be picked off at the start and the end of the day. The average of these two values is used as the average concentration for 17 July 1967. The tons of suspended sediment for 17 July would be computed as follows:

$$\text{Daily load} = 307 \text{ c.f.s. X } \frac{(1.7 + 1.1)}{2} \text{ G/L X } 2.7$$
$$\text{Daily load} = 1{,}160 \text{ tons}$$

For 17 June it would be necessary to divide the day into at least three eight-hour periods to have a representative average daily concentration. The average for the first eight-hour period would be 5.8 G/L, the second eight-hour period would be 6.0 G/L, and the last period would be 5.0 G/L. The daily average concentration would then be $\frac{5.8 + 6.0 + 5.0}{3}$ or 5.6 G/L. The suspended sediment load for 17 June would be computed as follows:

Daily load = 11,200 c.f.s. X 5.6 G/L X 2.7 = 169,344 tons

This method for computing daily suspended sediment load gives satisfactory results when sufficient samples have been taken to define the concentration curve. When samples are obtained at infrequent intervals good results can be obtained by trained personnel using the flow changes as a guide to the sediment concentration change. For a normal year, using daily average flow values, about two man-days per station would be required for hand computations. If only half of the 80 man-days could be saved with a computer program a substantial savings would be shown.

In developing our computer program, we approached it with the idea of finding a method of computing daily suspended sediment loads that would be comparable to that of hand computations. Lacking the intuition of the trained engineer, the computer results were found to be poor if we used daily average discharges. It was determined that much better accuracy could be developed by use of bihourly discharges. Bihourly flows gave satisfactory results; therefore, the discharges have not been broken down for any shorter time period. It was decided not to make this a predictive type program, but one that would be dependent on the input data. If the sample data were poor or inaccurate the results would also be poor and inaccurate. Several methods of computing bihourly concentrations were investigated. The approach, which will be described herein, was the one that we found which most closely reproduced the hand computation. Two basic assumptions are involved. The first is that at any particular instant, the relation between discharge and sediment concentration is linear. The second assumption is that the relation between sediment concentration per unit of flow and time is linear. The program, as developed, starts with one suspended sediment sample and then goes to the next sample, This is done through information generated by the samples, not by any preimposed condition built into the program that might misinterpret the sample data. Therefore, there must be a sample to start a period and one to end the period. This period could be of any length (although it is limited in the computer program). The first sample (starting sample) is plotted against the discharge for its bihourly time period. For each sample a curve is used that passes through the plot of the suspended sediment concentration and corresponding flow and the origin. Figure 3 represents the plot of sample No. 1 and No. 2. The equation for curve No. 1 and curve No. 2 in Figure 3 is as defined by equation 2.

Eq (2) $X = \text{slope} \cdot Y$

$$\text{slope} = \frac{\text{g/l of the sample}}{\text{c.f.s. for the time period of the sample}}$$

Equation for the first sample will be $X = \text{slope}_1 \cdot Y$. Equation for the second sample will be $X = \text{slope}_2 \cdot Y$.

The number of bihourly time intervals between the two samples is found. Then the suspended sediment concentration in grams per liter for each time period is solved by the following equation:

GL for time period A is equal to G/L for flow during period A as picked from curve 1 prorated (based on time) toward the G/L value given by curve 2 for the flow during period A.

A simplified form is shown in equation 3.

Eq (3) $G/L_A = Q_A \left[\frac{P}{N} \quad (S_2 - S_1) + S_1 \right]$

G/L_A = gram/liter of suspended sediment for a certain time period

Q_A = discharge for the particular time period

S_1 = slope of first curve (sample 1)

S_2 = slope of second curve (sample 2)

P = number of bihourly time periods from sample 1 to time period A

N = number of time periods from sample 1 to sample 2

Figure 4 is an example of two days with samples and bihourly discharge shown. In reference to Figure 4 the following computation would be made from sample one to sample two.

Sample one is 1.96 G/L and 2,200 c.f.s. with slope of the curve being .0008909. Sample two is 3.81 G/L and 2,669 c.f.s. with the slope of curve being .0014275. There are 13 time periods from sample 1 to sample 2. For time period 6, 23 June, and 2,423 c.f.s. the concentration for the time period would be computed as follows:

$$G/L = Q_A \left[(S_2 - S_1) \frac{P}{N} + S_1 \right]$$

$$G/L = 2,423 \left[(.0014275 - .0008909) \cdot 1/13 + 0008909 \right]$$

$$G/L = 2.2587$$

G/L for time period 7, 23 June, and flow of 2,538 c.f.s. would be

$$G/L = 2,538 \left[(.0014275 - .0008909) \quad 2/13 + .0008909 \right]$$

$$G/L = 2.4706$$

The computer program will follow through the sequence from time period one until the time period of the second sample is reached. One of the intermediate computations would be like that for bihourly period 1 on 24 June, flow of 2,553 c.f.s., as shown below.

$$G/L \text{ period 1, 24 June} = 2,553 \left[(.0014275 - .0008909) \quad 8/13 + .0008909 \right]$$

$$G/L = 3.1175$$

A plot of each of these computations is shown as a dot on Figure 4.

The actual suspended load computation is made as shown in equation 4.
Eq (4) bihourly load in tons

$$= \text{bihourly flow (c.f.s.)} \times \text{average suspended sediment concentration(G/L)} \frac{\times \text{(2 hrs/bihourly} \times \text{60 min/hr} \times \text{60 sec/min} \times \text{62.4 lbs/cu. ft. water)}}{\text{1,000 g/liter} \times \text{2,000 lbs/tons}}$$

or

= bihourly flow (c.f.s.) X average suspended sediment concentration(G/L)

X 0.225

These are tabulated and printed out as tons per day with the ability of the program to print out bihourly tons when the c.f.s. exceeds a specified flow during a day.

For comparing hand computations with computer results, the bihourly G/L, as computed by the computer sediment program, has been plotted as a heavy line on the hand computed plot shown in Fig. 5 and Fig. 6. The hand plot is the same as shown in Fig. 1 and Fig. 2. There is very little difference in results between the hand computation and the computer computation. The differences are felt to be within reasonable limits. Where there are discrepancies, it would be hard to say which curve is better.

Figures 7 and 8 are used to illustrate the effect on the computer method by limiting the number of samples. First, the curve is shown with all the available samples used in the computation, second with every other sample used, and third every fourth sample used of the available samples. So it may be seen that the more samples you use the better the results. Table 1 is a printout of a computer run for the station on Little Blue River near Barnes, Kansas, for the 1967 water year. The total tonnage for the year was 1,745,827 as compared to 1,625,605 computed by hand prior to running the data through the computer. This is within 7 percent of the hand computation results and is felt to be sufficient for our needs. Having covered the theory on which our program is based and the results as compared to hand computations, I would like to cover some other features of the program and data handling.

First of all, we have geared sediment collection and reporting data so it can easily be entered into the computer program. Fig. 9 is a sample of our data supplied by our sediment observers. The following data are filled out in the field by the observer; stream, identification number, and location. Stream and location are self explanatory. The USGS description of the station is used as stream and gage name. The identification number, for identification of samples in the laboratory, is assigned by our office and ranges from 1 to 100. When the suspended sediment sample is collected a data sheet is prepared listing the military hour, stage, station, or location on the bridge, depth of sample, field number, and water temperature. Only the field number and identification number are placed on the sample bottle. The field sheets and samples are picked up and taken to the laboratory. This sheet is then used to record the laboratory results. The completed sheet can then be given to a key punch operator for punching the required data for suspended computer program.

The data that are needed for the program are: month, day, year, military hour, and suspended solids in G/L. These data are reviewed for possible data errors before key punching. Having entered the data into the computer, the daily values are printed out in the form shown in table 1. Also included in the printout are sediment samples in order of occurrence, G/L for the sample, flow (c.f.s.) picked from the bihourly hydrograph corresponding to the samples, the slope of each sample equation, and the slope times drainage area.

Hopefully, a computer program may eventually be developed that would require a minimum of actual samples for input and compute within a reliable degree of accuracy daily suspended sediment concentration. In developing a program of this type, many variables would have to be considered and, hopefully, with our programs some of the more important ones can be reviewed by slope X drainage area (DA) shown in the printout (table 2).

There is quite a variance in slope/DA. column, but there should be some controlling factors such as, being the first, second, or third rise in flow during a particular period. Also, whether the sample is on the rising or falling portion of hydrograph should effect the slope values. Using the slope times drainage area will also allow comparison between sediment stations. These slopes could possibly be related as to soil types, relief, agricultural practices, and other physical parameters. With this type of investigation by the computer, the kind of analysis may be made that would be impossible to make by hand. After a few years of operation, an anlaysis of this type may be attempted using the compiled data.

In summary, this suspended sediment program is operable and being used on a 301 RCA computer. The results available have been compared to that of independent hand computations and are found to be within the reasonable range of accuracy. As additional stations are computed, possible additional analysis can be made for a predicting type of program. The operating time for computing stations for one year is 20 minutes. Normally, daily, monthly, and yearly suspended sediment loads are printed out. Bihourly load printout can be made when a specified c.f.s. value is exceeded during the day. This program is coded in Fortran II and can be obtained from the Kansas City District Office, Corps of Engineers.

TABLE 1

LITTLE BLUE RIVER NR. BARNES, KANS.

DAILY SUSPENDED SEDIMENT LOAD — WATER YEAR 1967

DAY	OCT.	NOV.	DEC.	JAN.	FEB.	MAR.	APR.	MAY	JUNE	JULY	AUG.	SEP.
1	58.	19.	26.	5.	172.	182.	69.	17.	263.	5179.	296.	51.
2	44.	17.	27.	4.	146.	135.	65.	12.	301.	4295.	203.	56.
3	43.	15.	32.	4.	131.	101.	64.	12.	1300.	4724.	190.	92.
4	40.	12.	35.	6.	131.	67.	58.	13.	2742.	3576.	193.	134.
5	38.	9.	33.	8.	156.	60.	53.	18.	14911.	1838.	133.	9597.
6	34.	7.	36.	6.	171.	62.	66.	29.	18174.	1462.	97.	8826.
7	30.	10.	53.	4.	132.	65.	81.	47.	14124.	1256.	118.	2476.
8	27.	13.	124.	2.	131.	64.	281.	58.	10644.	1167.	210.	882.
9	24.	14.	164.	4.	183.	65.	1071.	62.	118151.	1148.	349.	451.
10	21.	14.	95.	6.	221.	82.	1069.	55.	147574.	1397.	194.	234.
11	18.	14.	81.	6.	220.	108.	406.	46.	140083.	4933.	144.	132.
12	25.	14.	76.	8.	322.	112.	395.	40.	102604.	2426.	123.	82.
13	22.	15.	58.	9.	415.	79.	516.	34.	98365.	4928.	96.	92.
14	21.	15.	32.	25.	426.	62.	1287.	34.	88081.	2435.	85.	2276.
15	25.	17.	28.	41.	300.	45.	3311.	33.	38162.	1413.	66.	8605.
16	49.	15.	31.	54.	188.	41.	1868.	32.	174987.	1194.	62.	2997.
17	67.	13.	34.	83.	109.	41.	1010.	51.	180448.	967.	65.	1636.
18	91.	11.	32.	78.	82.	48.	724.	57.	128927.	808.	149.	1213.
19	93.	9.	31.	71.	68.	48.	626.	65.	24580.	681.	79.	2963.
20	84.	8.	32.	82.	55.	41.	407.	71.	10354.	447.	79.	13313.
21	73.	7.	27.	98.	42.	44.	142.	69.	8163.	271.	108.	18001.
22	66.	7.	18.	106.	61.	53.	53.	62.	7252.	152.	133.	16785.
23	57.	7.	12.	118.	83.	68.	41.	53.	14505.	133.	135.	7158.
24	51.	8.	10.	115.	156.	74.	32.	55.	27507.	168.	122.	3559.
25	44.	8.	6.	98.	315.	67.	21.	56.	33342.	153.	129.	1559.
26	36.	9.	4.	80.	336.	63.	20.	55.	41072.	187.	253.	988.
27	28.	10.	4.	64.	347.	71.	20.	57.	43298.	578.	339.	2104.
28	28.	9.	4.	82.	324.	60.	19.	59.	27602.	570.	192.	3247.
29	22.	10.	4.	101.		56.	20.	112.	22147.	650.	107.	1538.
30	20.	17.	4.	152.		55.	20.	124.	11614.	819.	55.	738.
31	19.		6.	162.		58.		163.		515.	51.	
	1298.	353.	1159.	1682.	5605.	2177.	13815.	1651.	1551277.	50470.	4555.	111785.

YEARLY TOTAL = 1745827. TONS

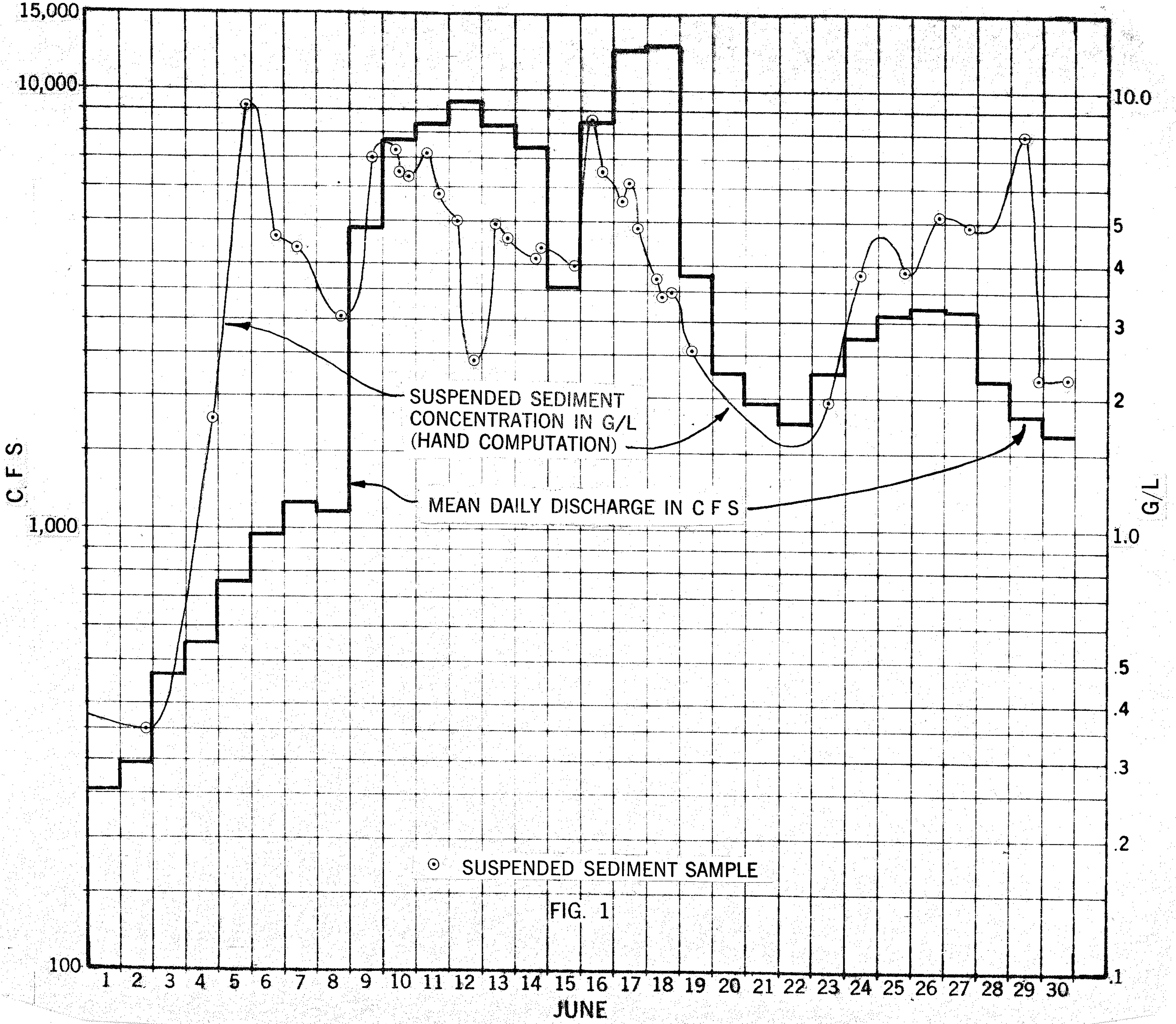
TABLE 2

LITTLE BLUE RIVER NR. BARNES, KANS.

SLOPE COMPUTATIONS

WATER YEAR 1967

DATE	TIME	G/L	DISCHARGE	SLOPE	SLOPE X D.A.	Q/D.A.
6.- 2.	1800.	.350	300.	.00116670	3.8780000	.09025271
6.- 4.	1800.	1.790	528.	.00339020	11.2688640	.15884477
6.- 5.	2100.	9.100	764.	.01191100	39.5921470	.22984356
6.- 6.	1800.	4.580	845.	.00542010	18.0164730	.25421179
6.- 7.	1000.	4.390	1152.	.00381080	12.6669790	.34657040
6.- 8.	1900.	3.030	990.	.00306060	10.1734550	.29783394
6.- 9.	1900.	6.990	4818.	.00145080	4.8224910	1.44945850
6.-10.	1000.	7.300	10592.	.00068920	2.2908990	3.18652230
6.-10.	1300.	6.480	8984.	.00072130	2.3975420	2.70276770
6.-10.	1700.	6.330	6177.	.00102480	3.4063330	1.85830320
6.-11.	1000.	7.150	9210.	.00077630	2.5805210	2.77075810
6.-11.	1800.	5.780	8600.	.00067210	2.2340370	2.58724430
6.-12.	900.	4.950	9278.	.00053350	1.7734210	2.79121540
6.-12.	1800.	2.370	9577.	.00024750	.8225830	2.88116730
6.-13.	1300.	4.910	8153.	.00060220	2.0018200	2.45276770
6.-13.	1900.	4.580	7978.	.00057410	1.9082380	2.40012030
6.-14.	1700.	4.130	7357.	.00056140	1.8659940	2.21329720
6.-14.	2100.	4.290	6122.	.00070080	2.3292980	1.84175690
6.-15.	1900.	3.990	2884.	.00138350	4.5987380	.86762936
6.-16.	900.	8.550	8950.	.00095530	3.1754410	2.69253910
6.-16.	1700.	6.450	8050.	.00080120	2.6633290	2.42178100
6.-17.	900.	5.640	10936.	.00051570	1.7142790	3.29001200
6.-17.	1200.	6.180	11900.	.00051930	1.7262450	3.58002410
6.-17.	1700.	4.860	14664.	.00033140	1.1016530	4.41155230
6.-18.	800.	3.710	15101.	.00024570	.8166370	4.54302050
6.-18.	1300.	3.430	13300.	.00025790	.8572420	4.00120340
6.-18.	1800.	3.480	11450.	.00030390	1.0102640	3.44464500
6.-19.	900.	2.570	3881.	.00066220	2.2011540	1.16756920
6.-23.	1000.	1.960	2200.	.00089090	2.9613820	.66185319
6.-24.	1100.	3.810	2669.	.00142750	4.7450130	.80294826
6.-25.	2000.	3.870	3172.	.00122010	4.0554480	.95427196
6.-26.	2000.	5.160	3272.	.00157700	5.2420050	.98435620
6.-27.	2100.	4.980	3172.	.00157000	5.2186380	.95427196
6.-29.	1000.	7.830	2738.	.00285980	9.5058140	.82370638
6.-29.	1800.	2.220	1544.	.00143780	4.7793260	.46450060
6.-30.	1900.	2.200	1408.	.00156250	5.1937500	.42358604
7.- 4.	1900.	1.450	648.	.00223770	7.4379630	.19494585
7.-16.	2000.	1.410	303.	.00465350	15.4681190	.09115524
7.-20.	900.	.680	260.	.00261540	8.6935380	.07821901
7.-22.	1900.	.200	206.	.00097090	3.2271850	.06197353
7.-28.	1100.	.920	223.	.00412560	13.7133630	.06708785
7.-30.	1000.	.870	370.	.00235140	7.8158920	.11131167



15,000
10,000
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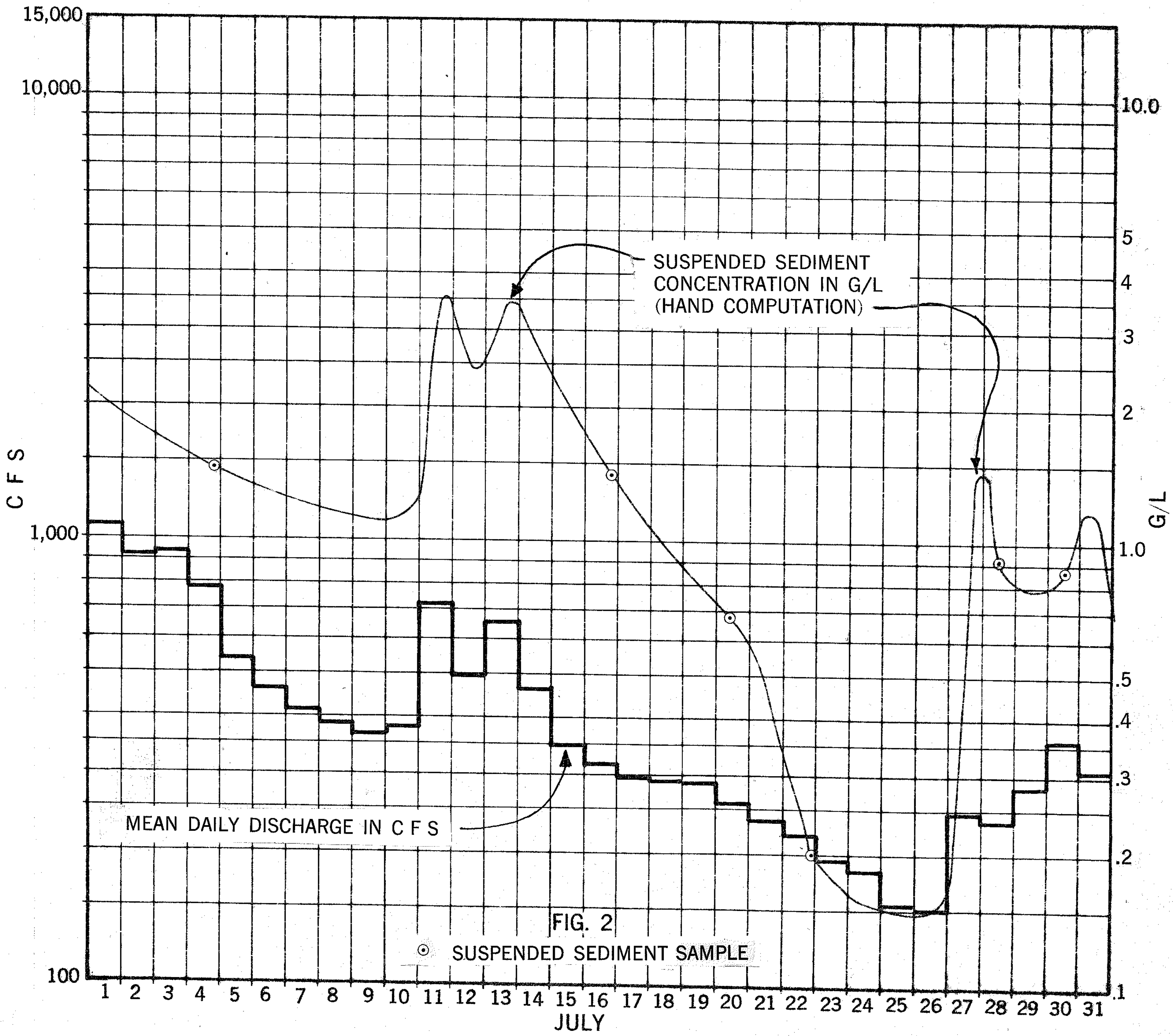
G/L
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
JUNE
SUSPENDED SEDIMENT CONCENTRATION IN G/L (HAND COMPUTATION)
MEAN DAILY DISCHARGE IN C F S
SUSPENDED SEDIMENT SAMPLE

FIG. 1



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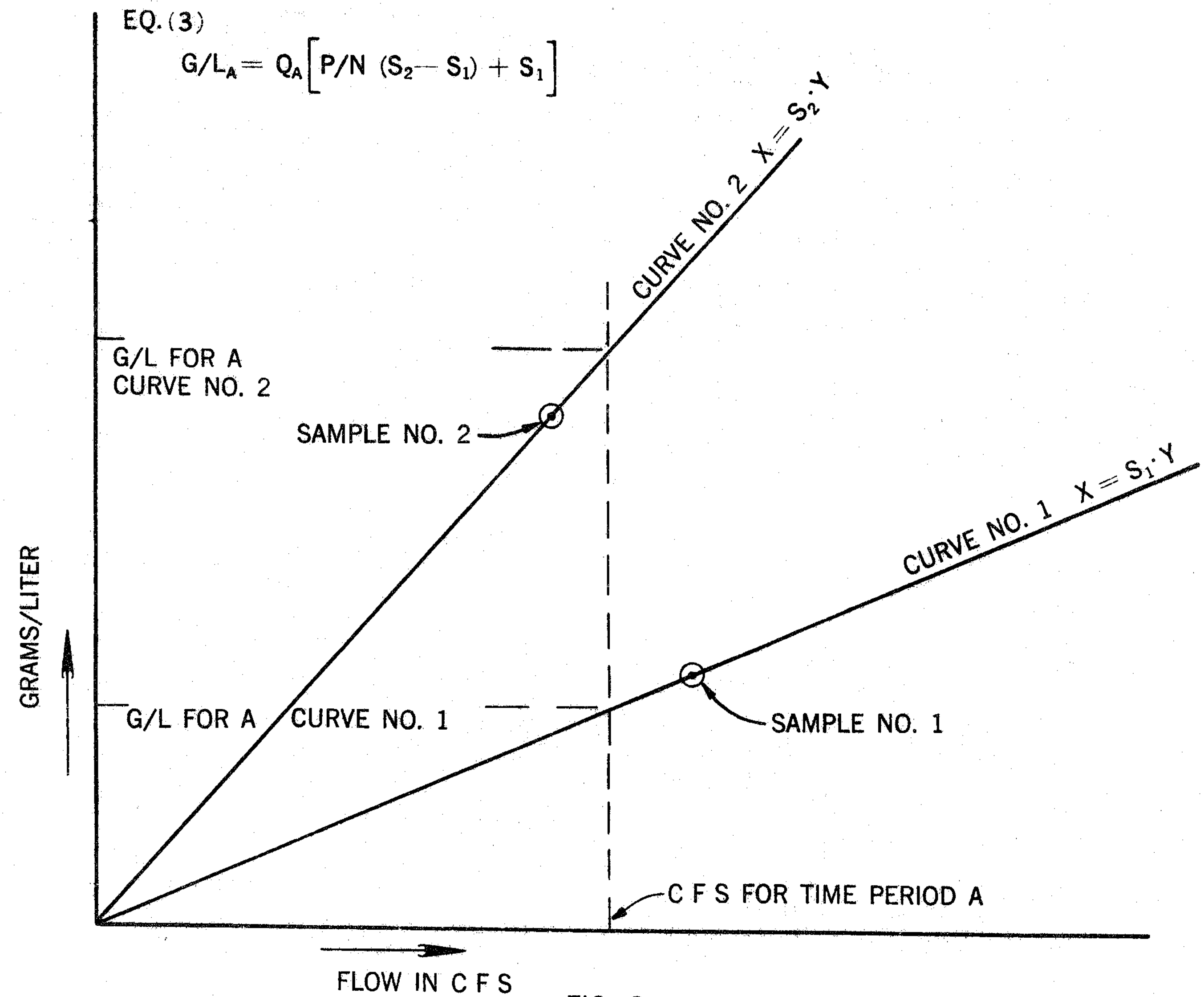
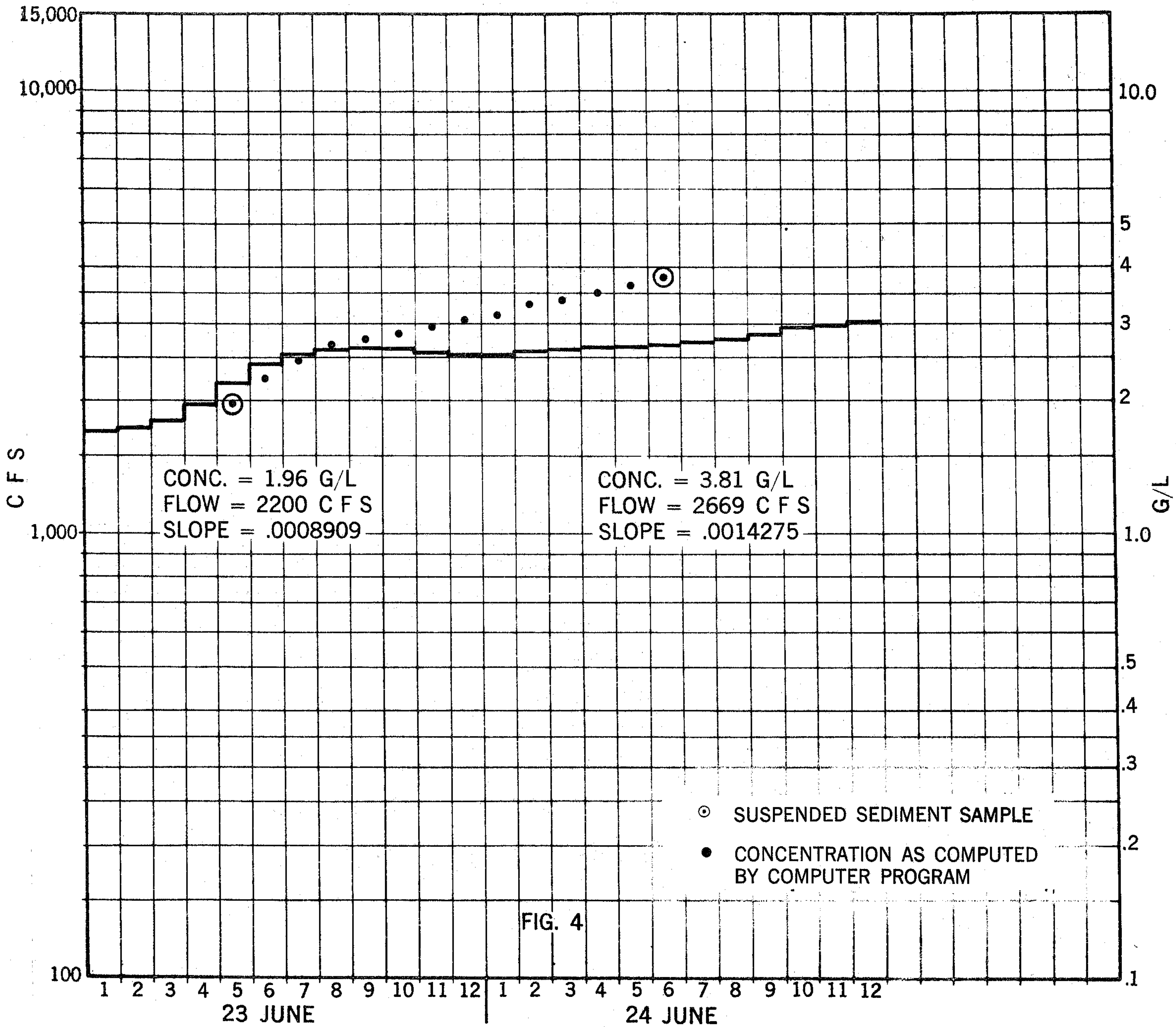
G/L
SUSPENDED SEDIMENT
CONCENTRATION IN G/L
(HAND COMPUTATION)
MEAN DAILY DISCHARGE IN C F S
SUSPENDED SEDIMENT SAMPLE
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
JULY

FIG. 2



EQ. (3)
$G/L_A = Q_A\left[P/N\ (S_2 - S_1) + S_1\right]$
GRAMS/LITER
G/L FOR A CURVE NO. 2
SAMPLE NO. 2
CURVE NO. 2 $X = S_2 \cdot Y$
G/L FOR A CURVE NO. 1
SAMPLE NO. 1
CURVE NO. 1 $X = S_1 \cdot Y$
C F S FOR TIME PERIOD A
FLOW IN C F S

FIG. 3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100
C F S
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2
1.0
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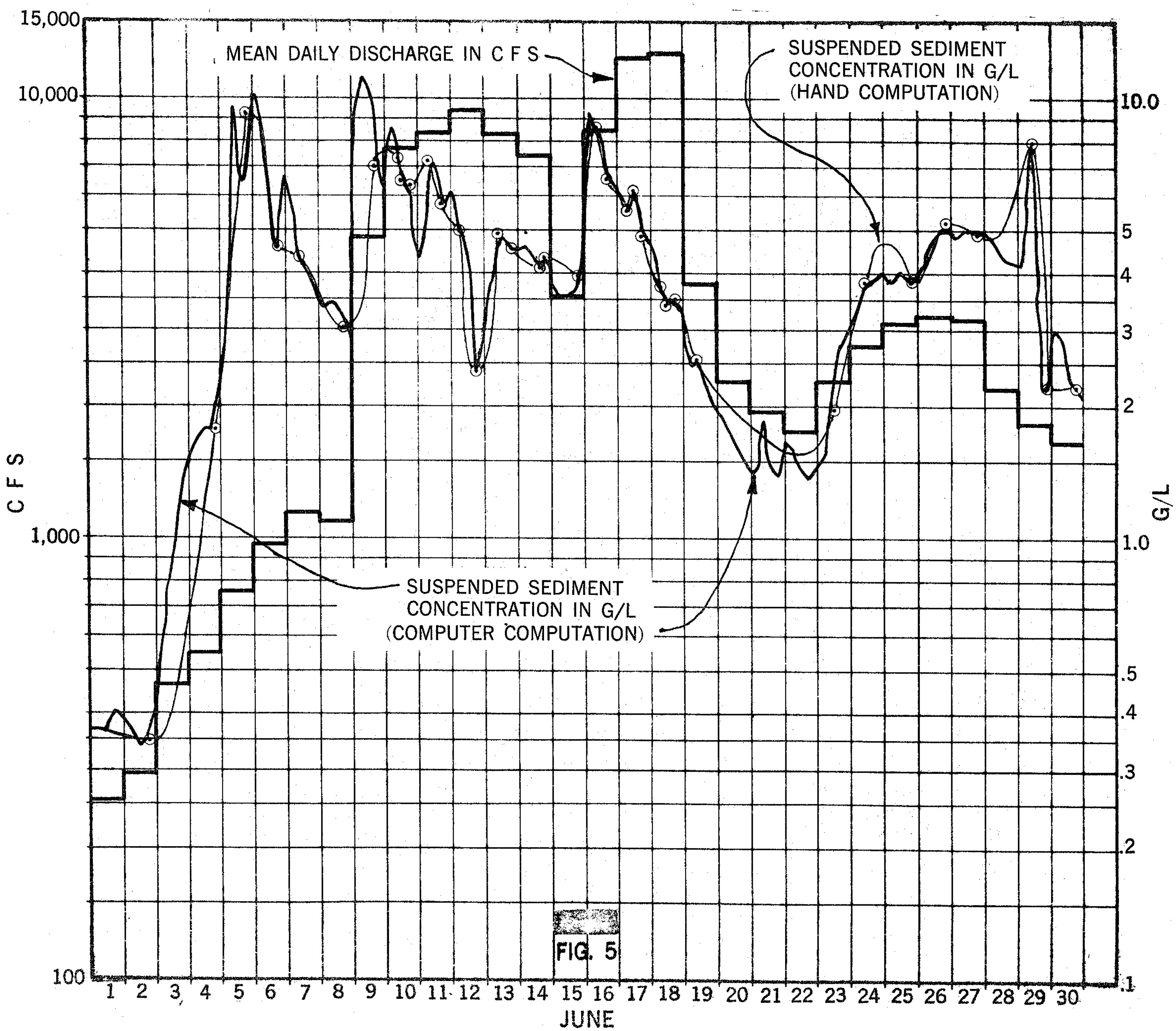
.3
.2
.1
G/L
CONC. = 1.96 G/L
FLOW = 2200 C F S
SLOPE = .0008909
CONC. = 3.81 G/L
FLOW = 2669 C F S
SLOPE = .0014275
SUSPENDED SEDIMENT SAMPLE
CONCENTRATION AS COMPUTED BY COMPUTER PROGRAM
1 2 3 4 5 6 7 8 9 10 11 12 1 2 3 4 5 6 7 8 9 10 11 12
23 JUNE
24 JUNE

FIG. 4


MEAN DAILY DISCHARGE IN C F S
SUSPENDED SEDIMENT CONCENTRATION IN G/L (HAND COMPUTATION)
SUSPENDED SEDIMENT CONCENTRATION IN G/L (COMPUTER COMPUTATION)
C F S
15,000
10,000
1,000
100
G/L
10.0
5
4
3
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1.0
.5
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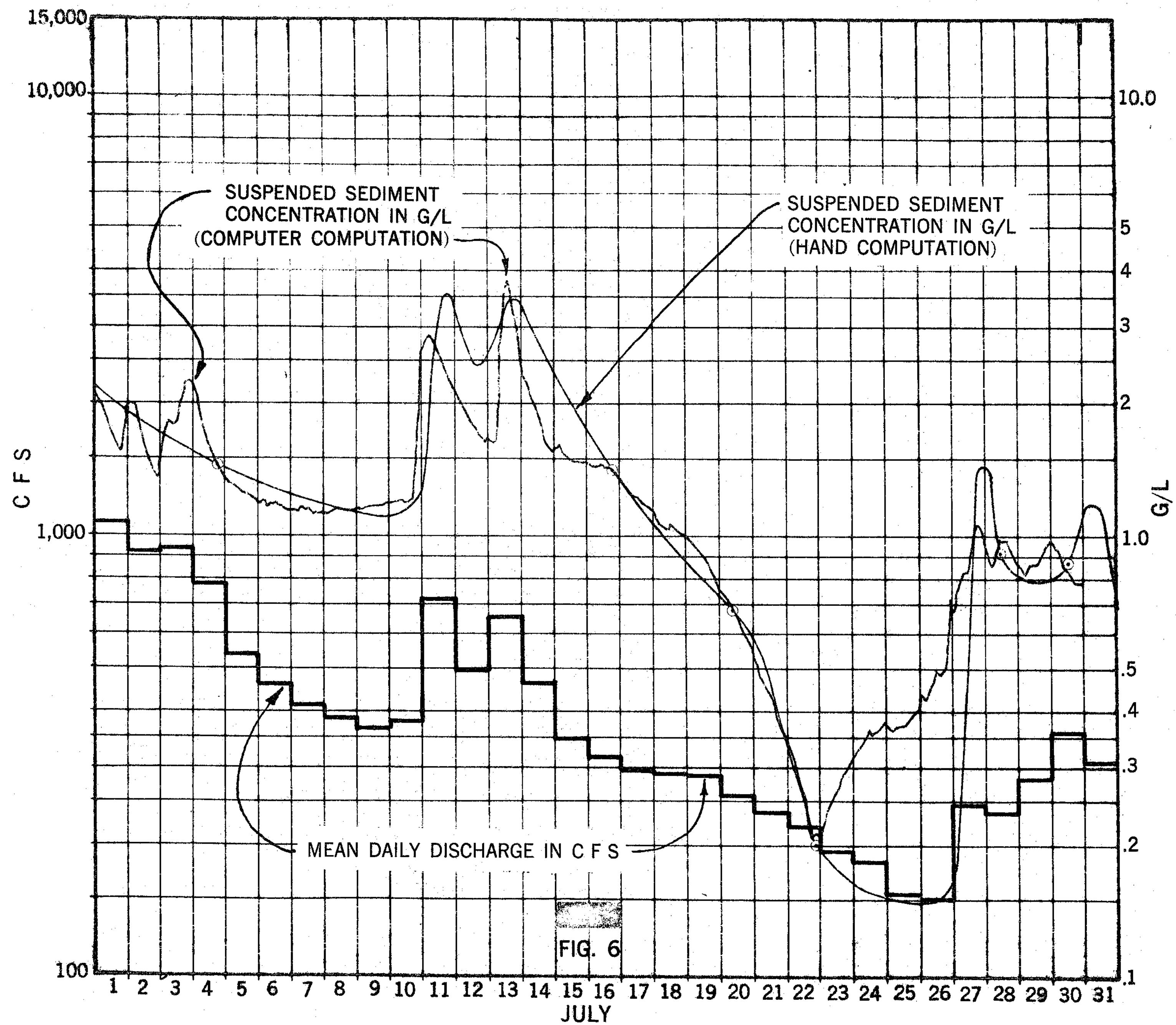
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
JUNE

FIG. 5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C F S
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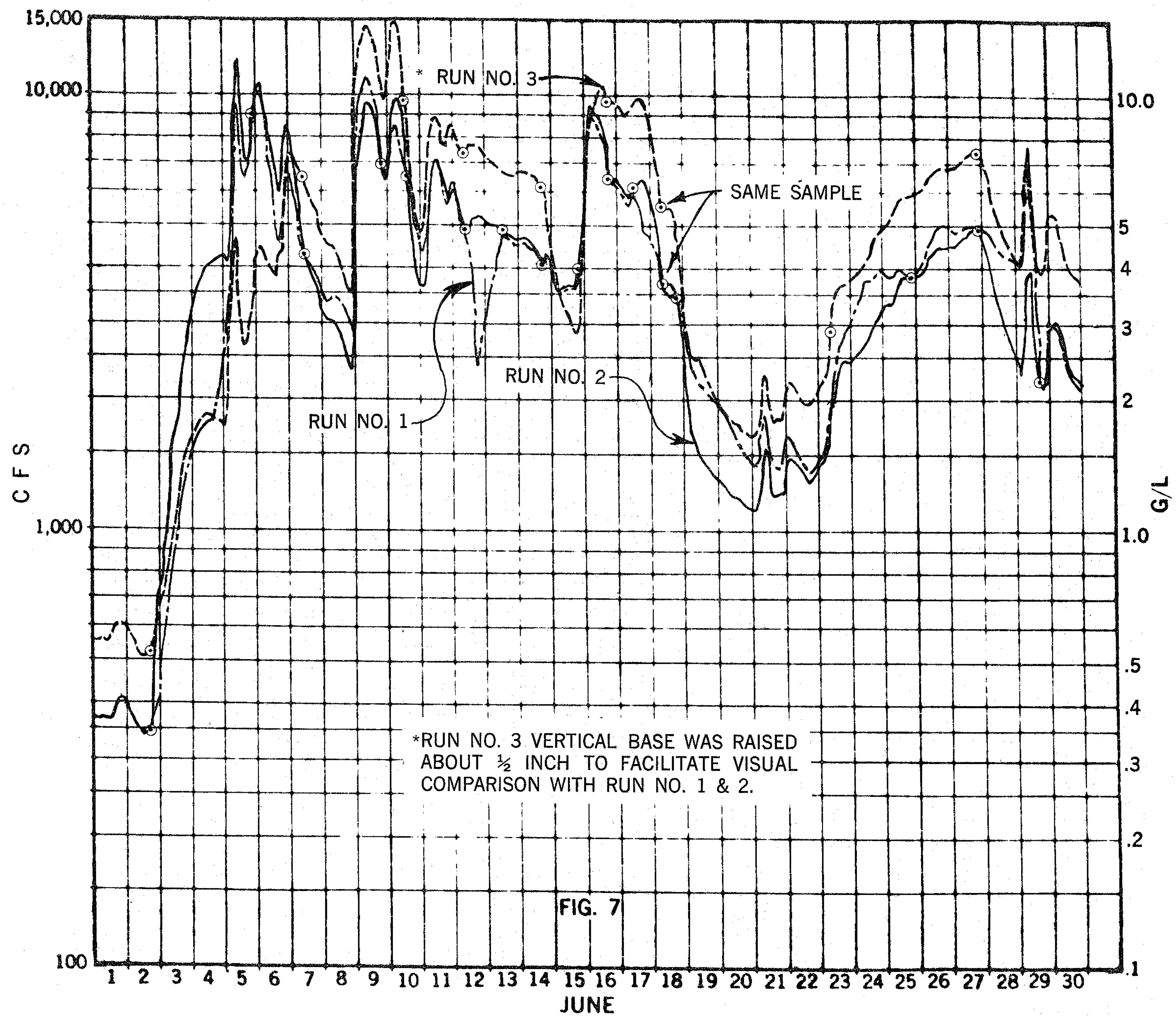
.1
G/L
SUSPENDED SEDIMENT CONCENTRATION IN G/L (COMPUTER COMPUTATION)
SUSPENDED SEDIMENT CONCENTRATION IN G/L (HAND COMPUTATION)
MEAN DAILY DISCHARGE IN C F S
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
JULY

FIG. 6


15,000
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100
C F S
10.0
5
4
3
2
1.0
.5
.4
.3
.2
.1
G/L
* RUN NO. 3
SAME SAMPLE
RUN NO. 1
RUN NO. 2
*RUN NO. 3 VERTICAL BASE WAS RAISED ABOUT ½ INCH TO FACILITATE VISUAL COMPARISON WITH RUN NO. 1 & 2.
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
JUNE

FIG. 7

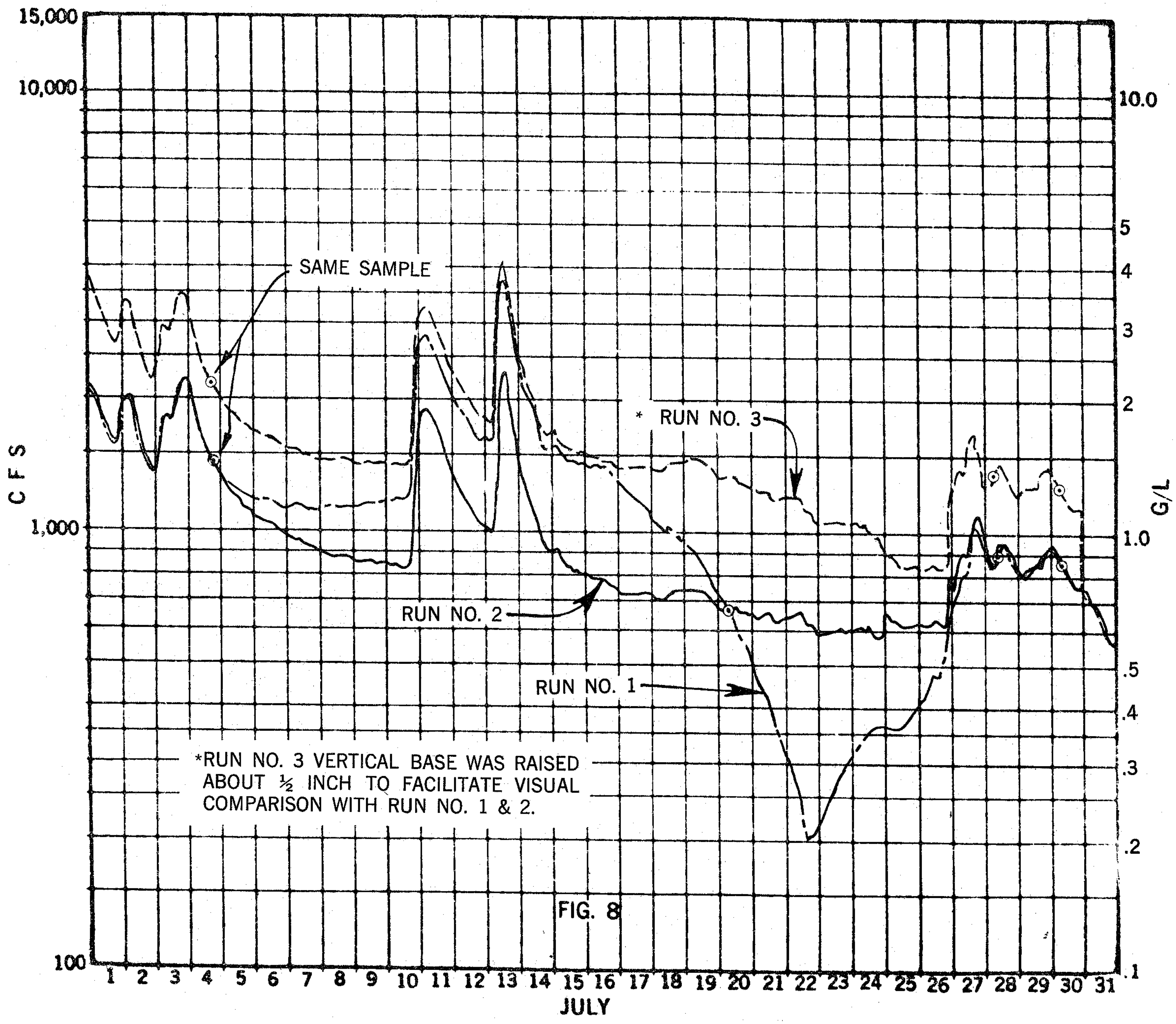
15,000
10,000
1,000
100
C F S
10.0
5
4
3
2
1.0
.5
.4
.3
.2
.1
G/L
SAME SAMPLE
* RUN NO. 3
RUN NO. 2
RUN NO. 1
*RUN NO. 3 VERTICAL BASE WAS RAISED ABOUT ½ INCH TO FACILITATE VISUAL COMPARISON WITH RUN NO. 1 & 2.
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
JULY

FIG. 8

FIG. 9

SEDIMENT SAMPLE RECORD

Corps of Engineers
Kansas City District

STREAM: Little Blue ID NO: 72

Location: Barnes, Kansas

Date Mo. Day Yr.	Military Hour	Stage	Sta.	S'dg.	Field No.	Solid G/L	Solu. G/L	%over .062mm	Temp F	Conduct-ance
COL. 1-8	COL. 10-13					COL. 16-20				
6-2-67	18:00	3.53		2.3	46	0.35	0.30		71°	
6-4-67	18:00	4.19		3.1	47	1.79			80°	
6-5-67	21:30	5.52		4.1	48	9.10			70°	
6-6-67	18:30	4.58		4.1	49	4.58			74°	
6-7-67	10:00	5.53		4.4	50	4.39			74°	
6-8-67	19:30	5.23		4.6	51	3.03			78°	
6-9-67	19:00	10.31		9.6	52	6.99			70°	
6-10-67	10:00	15.90		14.6	53	7.30	0.30		66°	

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20 Mar. 67

CURRENT DUTCH PRACTICE
FOR
EVALUATING RIVER SEDIMENT TRANSPORT PROCESSES

by

Helmer O. Johnson[1]

INTRODUCTION

Man made intervention in natural river systems will result in changes of sediment transport, scour and deposition. Knowledge of the mechanism of sediment transport will give insight into the expected results of artificial intervention and will decrease the chance of failure.

This paper is intended to survey the procedures relative to river processes and sediment transport which are currently being used in Europe and particularly in the Netherlands.

SEDIMENT TRANSPORT

General

Although insight and knowledge of water discharge in a river is undoubtedly important for various purposes, the formation and deformation of the river bed cannot be considered without a study of sediment transport. Sediment transport is the main factor which determines the behavior of the bed and boundary conditions.

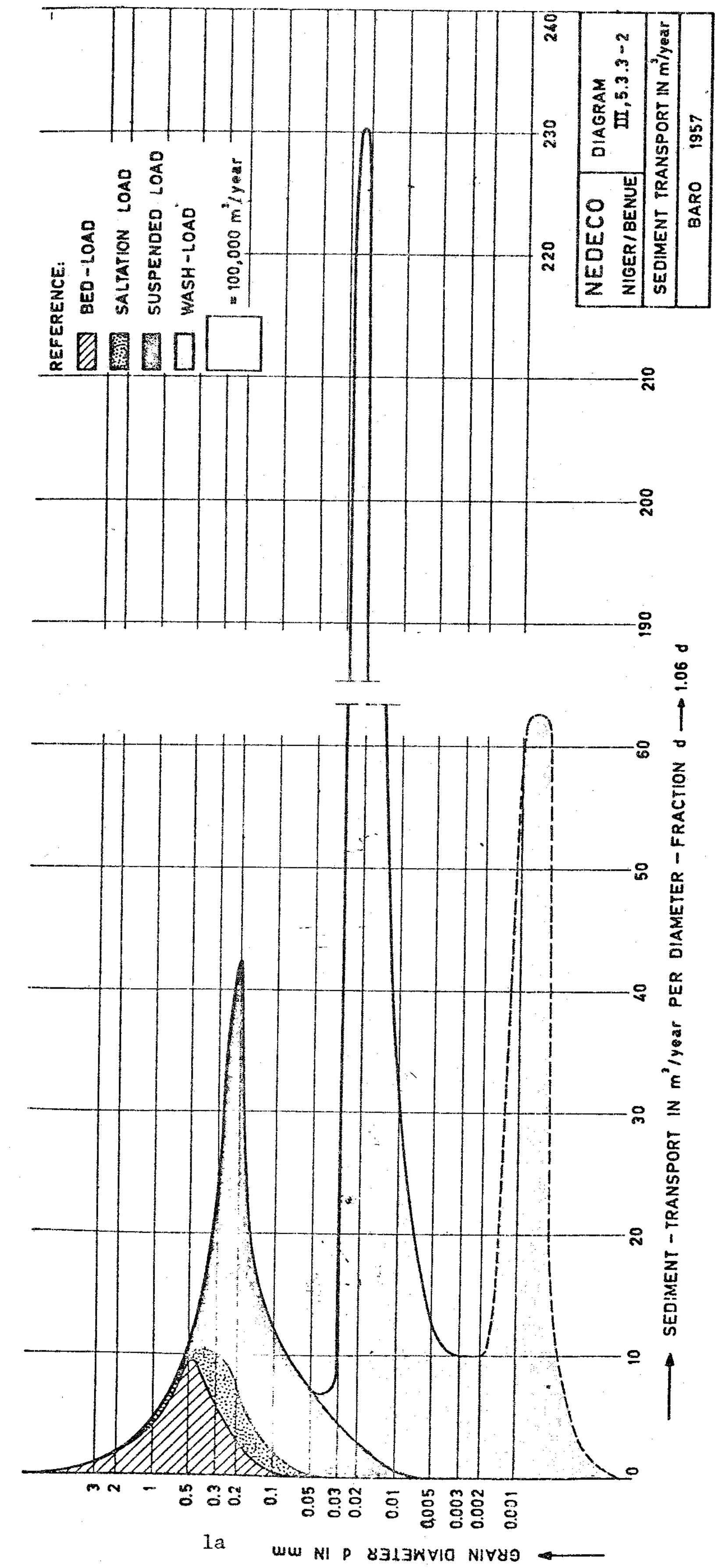
Wash Load

Wash load is composed of the finest particles of silt and other matter that are brought into the river and remain suspended until they reach the sea or perhaps are deposited in some depression. Wash load has very little influence upon the river's behavior, although it may quantitatively be the largest amount of sediment transported by the river (see Fig. 1). The largest quantities of wash load are found during the rise of the river which implies that there is not a perfect correlation between wash load and water discharge (Q). The quantity carried depends upon the supply rather than the carrying capacity of the flow.

Suspended Sediment

Suspended sediment consists mainly of fine sand grains that are continuously supported by the turbulent action of the flow. Over a short stretch of river the suspended load can be assumed to remain in suspension. This type of transport has little influence upon the river bed.

[1]Chief, Reservoir & Hydrologic Engineering Section, St. Paul District


REFERENCE:
BED-LOAD
SALTATION LOAD
SUSPENDED LOAD
WASH-LOAD
= 100,000 m³/year
NEDECO
DIAGRAM
NIGER/BENUE
III, 5.3.3 - 2
SEDIMENT TRANSPORT IN m³/year
BARO
1957
SEDIMENT - TRANSPORT IN m³/year PER DIAMETER - FRACTION d ⟶ 1.06 d
0
10
20
30
40
50
60
190
200
210
220
230
240
3
2
1
0.5
0.3
0.2
0.1
0.05
0.03
0.02
0.01
0.005
0.003
0.002
0.001
GRAIN DIAMETER d IN mm

1a

Figure 1

There is no unique correlation between Q and suspended load; however, the peak of the suspended load will probably coincide with the peak of the water discharge hydrograph.

Bed Load Sediment

Bed load sediment (T) refers to the larger grains of material carried by the river that roll, slide, and jump along the bottom but hardly rise from the bottom. Their movement extends some distance into the bed with an exchange of grains from different layers. Even though this type of sediment comprises the smallest quantity in the total transport, it is by far the most important type.

The transporting capacity is always fully satisfied because the bed is an unlimited source of supply of material. Two consequences of this are:

a. There is a definite correlation between the discharge (Q) and bed load transport (T).

b. When the drag force decreases for one reason or another, the bed load transport (T) decreases which immediately affects the bed.

BED LOAD FORMULAS

General

Because of the difficulty in obtaining accurate bed load measurements by instrument, preference is often given to the computation of sand transport. Various bed load transport formulas have been developed, but it appears that for conditions that are common in normal rivers, all the formulas give more or less the same quantitative result (see Fig. 2). Various authors have used different notations, but all formulas can be modified to involve some relation between two dimensionless parameters, one representing the bed load transport (T) and the other the drag force.

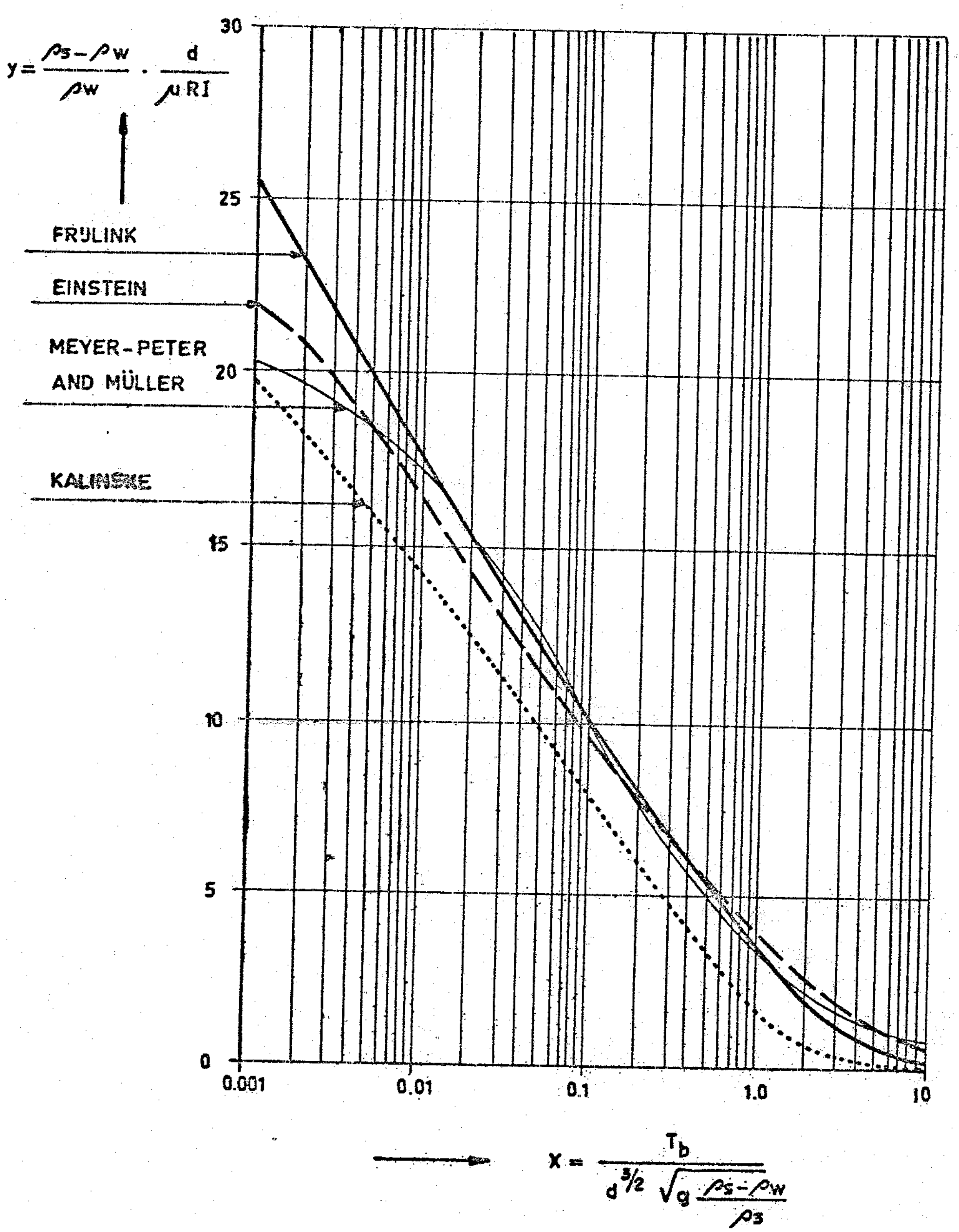
Formulas Presently Used by the Dutch

Dutch engineers use the Meyer-Peter and Muller bed load formula expressed below.

Equation #1:

$$T = 8bd^{3/2} \cdot \sqrt{g\Delta}\left[\frac{\mu hi}{\Delta d} - 0.047\right]^{3/2}$$

PROPOSED EQUATION BY FRIJLINK: $X = 5y^{-1/2}e^{-0.27y}$



COMPARISON OF FORMULAE OF KALINSKE, EINSTEIN, MEYER-PETER AND MULLER, AND FRIJLINK.

Figure 2

where b = width of river channel
d = diameter of bed material (d_m size)
g = acceleration of gravity
$\Delta = \frac{\rho\ \text{sand} - \rho\ \text{water}}{\rho\ \text{water}}$
ρ = density
μ = ripple factor
h = depth of flow in river
i = slope of water surface

Analytically and mathematically it is the easiest formula to work with. The term 0.047 used within the formula apparently represents a kind of critical value for the drag force below which no bed load transport takes place. This value is illustrated on Shield's graph (see Fig. 3).

RIVER MORPHOLOGY

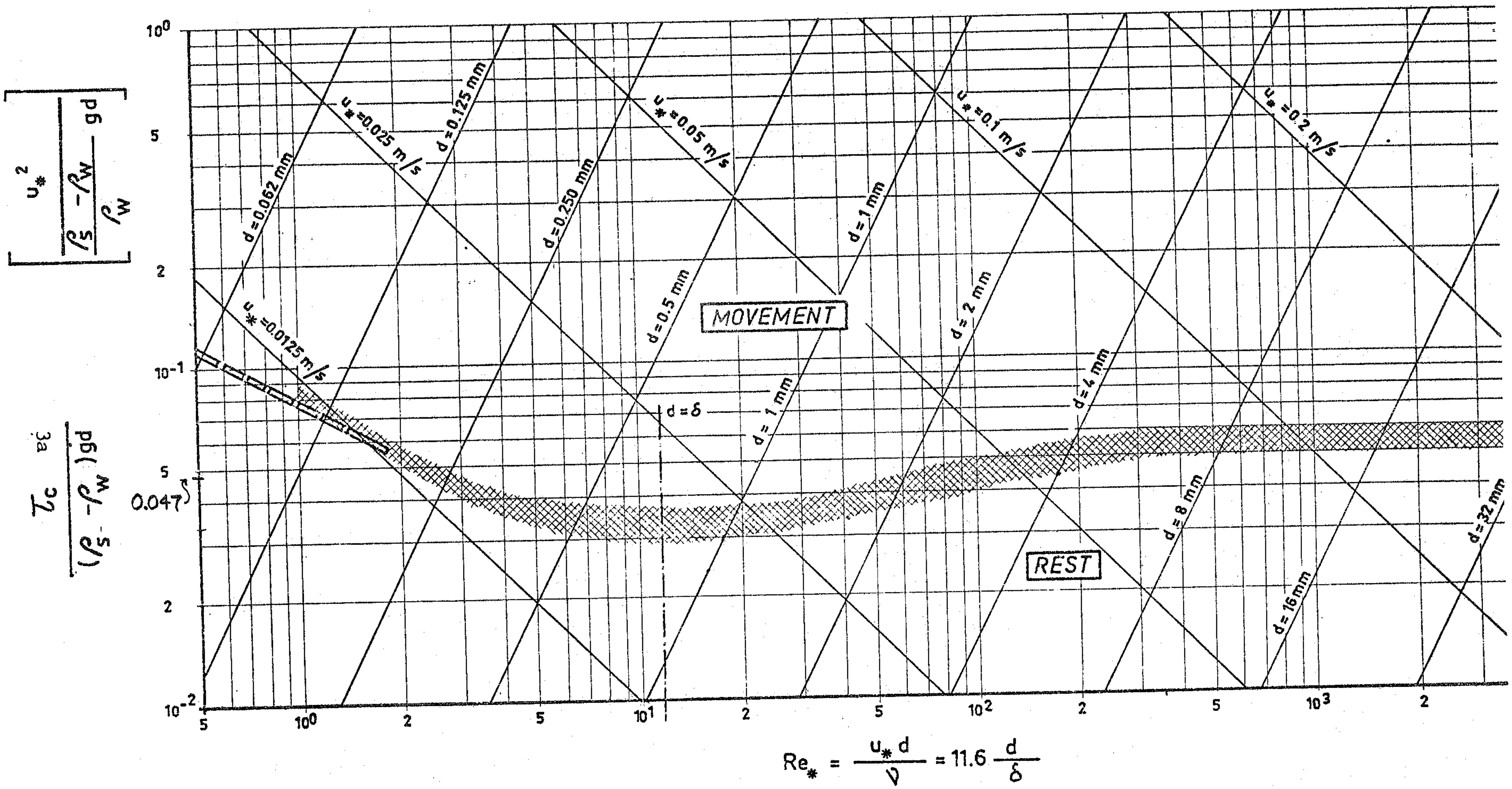
Independent and Dependent Variables

In an alluvial river, the valley is to a large extent created by the river itself. Assuming this is true, there must be a relationship between the two principle tasks of the river, (its flow discharge Q and its sediment transport T) and all the characteristics of the riverbed. There is a third property of the river which must also be considered, namely, the bed sediment size (d). This may not be as important as Q or T because it is subject to the river's influence. The three independent variables are then Q, T, and d.

Dependent variables are the width of the channel (b), the depth (h), the slope of the water level (i), and the roughness of the bed, Chezy's (C) or ripple factor (μ). The knowledge of morphology has been developed to a point where if the independent variables are known, it is possible to give a fair indication of the values for the dependent variables. See following paragraphs for details.

Other dependent variables, for which quantitative functions and correlations have not yet been solved, include meandering, island formation, and **branching tendency.**

Sometimes laboratory tests are used to obtain solutions to these river hydraulics problems, and much information has been obtained this way. However, a word of warning seems justified. Laboratories work with flumes that in most cases are tilted so as to give a constant slope. Independent



LINES OF EQUAL u_* AND d BASED ON ρ_s = 2650 kg/m^3 AND ν = 1.25 × 10^{-6} m^2/s (12 °C)

RELATIONSHIP OF CRITICAL SHEAR STRESS AND DIAMETER FOR A BED OF UNIFORM GRAINS. ACC. TO SHIELDS

Figure 3

variables in this case are Q, d, and the slope (i), whereas T is reduced to a dependent magnitude. The consequences of this reversal of functions is not fully understood, but it seems wise to regard with care results obtained in such flumes.

Four Basic Equations Needed to Define a River

Assuming that nature will mold the river according to its natural laws, it should be possible to determine values for the dependent variables when the independent variables are known. There are three basic equations which can be used to make this determination; Chezy's formula, the formula for bed load (say Meyer-Peter and Muller), and experimental formula giving the roughness of the bed, either C or μ. A fourth equation is needed in order to define the four independent variables.

Least Energy Consumption Hypothesis

It seems logical to assume that a river will flow under the condition of least resistance, or more accurately, "least energy consumption" 4/ which means "least slope i, just capable of transporting the given quantity T".

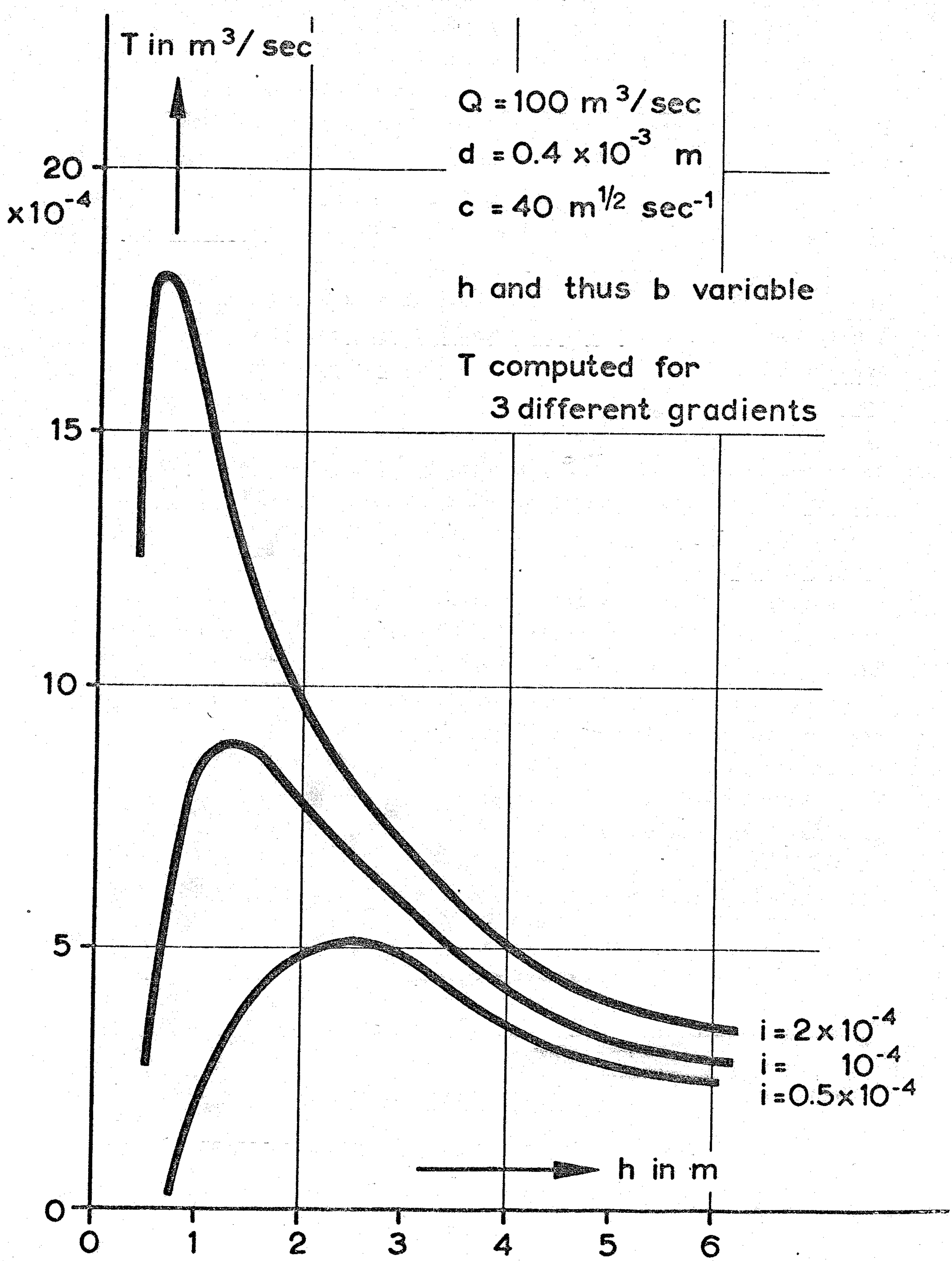
This can be better explained by an example. If, for a certain constant Q, T is computed for a variety of profiles ranging from a wide and shallow cross section to a narrow and deep one, it appears that the function T = f(h) takes the form of a curve with a definite maximum value. Curves of T versus h for different values of i are shown in Fig. 4. From the figure it appears that in order to transport a certain given T, there is a minimum slope required below which the target cannot be achieved. It appears that this situation, reflecting the most efficient profile capable of complying with nature's law, with respect to T, Q and d, will be the equilibrium profile of the river. This hypothesis is still not proved, but it appears logical that nature will strive for the optimum use of its available energy. Stated in another way, nature will try to achieve maximum sediment transport for a given limited available energy.

By applying the formulas of Chezy and Meyer-Peter and Muller, and introducing this least energy consumption hypothesis as the third condition, it will yield a complicated differential equation which reduces to a simple mathematical form:

Equation #2:

$$\frac{\Delta d}{\mu h i} = \text{constant}$$

This condition is valid only for equilibrium situations, or for "dominant discharge" 3/ 4/ 6/.



VARIATIONS OF TRANSPORT CAPACITY WITH DEPTH AND SLOPE

Figure 4

Some consequences of equation # 2 are listed below:

a. The depth is independent of the discharge. This has been proved by model experiments.

b. The depth would be inversely proportional to the slope. Deep and narrow channels develop in the lower reaches of rivers where slopes are gentle and wide and shallow channels develop where slopes are steep (h is proportional to d/i).

c. The slope i will be governed mainly by the ration T/Q (T/Q is proportional to i).

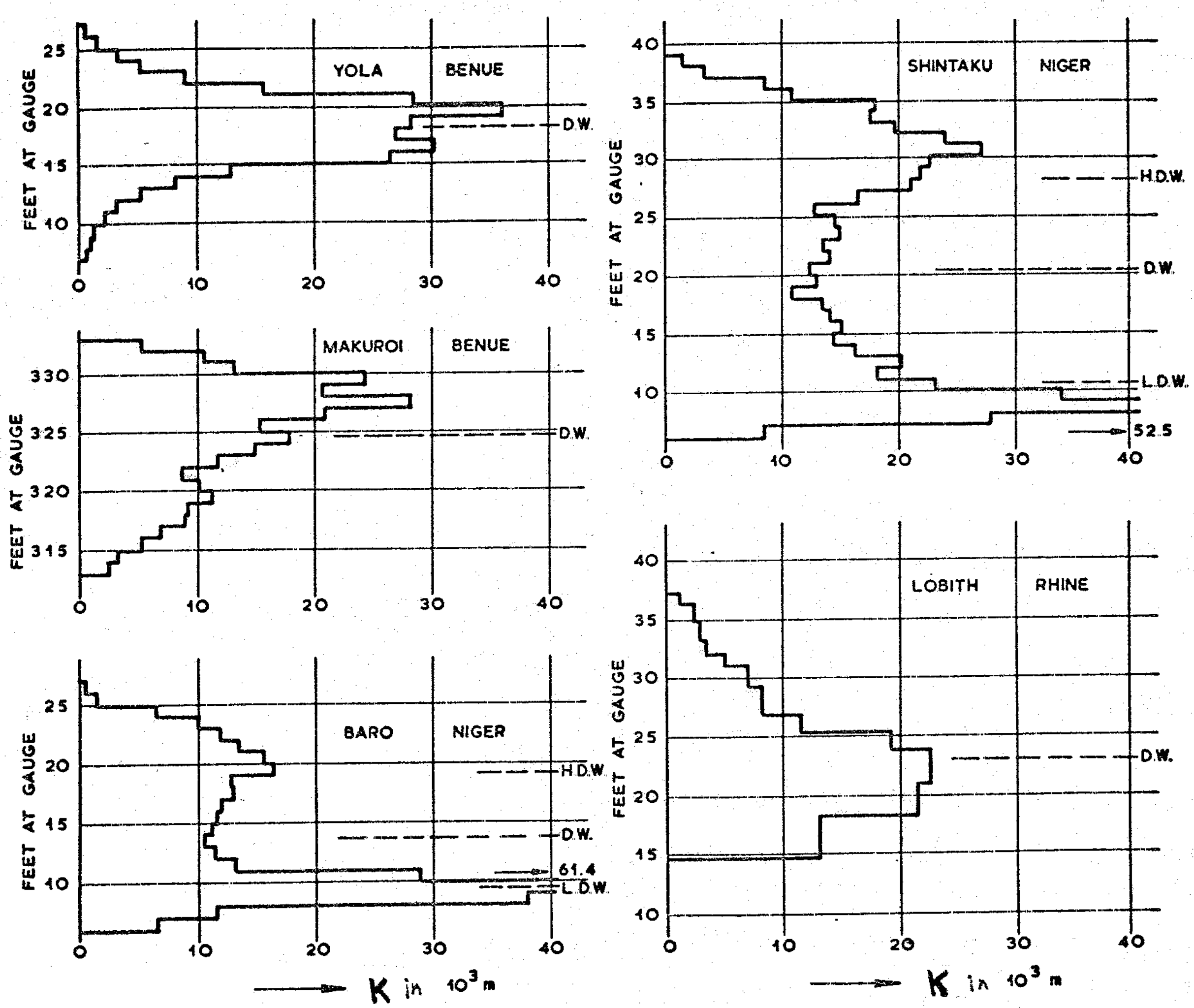
d. The ratio T/Q decreases in the downstream direction thus explaining why the slope of a river gradually decreases from headwater to delta.

DOMINANT DISCHARGE

General

It was stated earlier that with a certain Q, T, and d, the other river characteristics can be determined. However, the theories were originated assuming Q and T constant, i.e., the hydrograph was taken to be a horizontal line. There is no river in the world which complies with this condition. In most rivers, the Q and T vary considerably over the year. This implies that the other (dependent) characteristics of the river should vary. This is physically not possible. The transporting capacity of the river has limitations which would not allow deforming of the bed quickly enough to keep pace with the variations in the two independent variables Q and T. The most rigid of the four dependent variables is i. It would take a massive displacement of sediment to alter the slope. This may be possible after several years, but not within one year. The river width b is also more or less fixed. The depth h, however, is subject to rapid changes by the process of scour and deposition.

It appears, therefore, that i and b will be adopted to an average Q, T, and d, that h will vary around an average value, and that C will vary, keeping pace with flow conditions. By starting from the assumption that i and b do not vary in time, and using the formulas of Chezy and Meyer-Peter and Muller, the morphological equilibrium can be derived from the condition that the yearly scour and deposition should total zero. This principle can be applied to the full hydrographic period by dividing the Q-scale into intervals. The influence of a group of Q's within a certain interval (for which it is easier to use the h-scale instead of the Q-scale) can be mathematically expressed by the following equation:



Dominant water-levels along Niger, Benue and Rhine

Figure 5

Equation #3:

$$K = \frac{mTp}{h \cdot \Delta h} \qquad \text{(See App. II)}$$

Where m = number of days in Δh interval
T = bed load transport
p = coefficient (see Appendix II)
h = depth of flow in river
h = interval on h scale

The top part of the expression for K, that is in mTp, indicates the total transport during the period that the river stage is within the particular interval. Plotting K against the h-axis will result in a so-called K-diagram (see Fig. 5). The center of gravity of this diagram indicates a representative condition called "dominant condition". This dominant water level can be translated into dominant discharge by means of the Q-h rating curve. From the above, and information contained in the references, we can conclude that it is this dominant discharge that molds the riverbed. With this representative discharge, the previously derived equilibrium conditions are valid. The i, b and average h can now be computed.

A procedure for computing the dominant discharge by means of electronic computer can be developed.

RIVERBED FORM

Crossings

In a meander bend of a river, the water flows along alternating spirals resulting in a transversal flow along the bottom toward the inner bend. This implies that the cross section in a bend is triangular in shape. In the transition zone between a right bend and a left bend the cross section takes the form of a trough with a flat bottom. The thalweg must cross from the one outer bank to the other, passing a location where the depth is shallower. This sill is called a "crossing" and can form a bottleneck for navigation.

Bed Scour and Deposition

High flows will tend to scour the bends of a river and cause deposition in the crossing area. Low flows will tend to reverse the process, although this is a much slower process. Any reduction of the high flows, due to construction of a reservoir for example, will improve the channel for navigation. This will also reduce the dominant discharge and effect the characteristics of the river.

Q-h Relation

The loop effect on a rating curve is a well known phenomenon. Many different influences such as overbank storage and temperature of the water, for example, can affect the rating curve. Many European engineers prefer the following general explanation for this phenomenon. As the river rises, it causes bigger bed-ripples (or dunes) to form, but there is a time-lag in this ripple formation so that the reverse procedure is slower. Chezy's C is, therefore, high during rising stages and low during falling stages. It must be borne in mind that in the case of a large flat flood plain the opposite may occur.

SUMMARY

The dominant discharge procedure enables engineers to determine the bed forming discharge of a river and also can serve as a guide in the design of river groynes (wing dams). Dutch engineers recommend that the height of groynes be governed by the dominant water level (dominant discharge) so as not to interfere with flood discharges.

It is apparent that sediment transport and particularly bed load transport (T) is very important to the formation and character of a river. It is hoped that this paper has given insight into the various aspects related to river engineering and perhaps indicated solutions to some river problems. It must be borne in mind that the ideas presented in this paper are currently held by the Dutch and many other European engineers, but may not necessarily be consistent with current views of engineers in the U.S.

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4. H. A. Berdenis van Berlekom — "The Role of Rivers to Mankind" - Presented at the Polish Acadamy of Sciences Institute of Hydro-Engineering, Gdańsk - June 1969.

5. Dr. M. de Vries — "Solving River Problems by Hydraulic and Mathematical Models" Delft Hydraulics Laboratory, Delft, Holland.

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APPENDIX I - Symbols

Q	water discharge
T	bed load transport
d	diameter of the bed load material (d_m size)
b	width of the river
h	depth of flow in river
i	slope of the water surface
C	Chezy's coefficient
μ	ripple factor
k_s	sand grain roughness (dia. of grain d_{90} size)
k_r	actual bed roughness
ρs	density of sand
ρw	density of water
n	Manning's roughness coefficient
R	hydraulic radius
$\overline{u}$	average velocity
Δh	interval on h scale
g	acceleration of gravity
m	number of days in Δh interval
c	critical shear stress
u^*	shear velocity
Re^*	Reynolds Number
ν	kinematic viscosity
δ	thickness of laminar sublayer

APPENDIX II - Basic Formulas

Chezy -

$$\bar{u} = C\sqrt{Ri}$$

$$C = 5.75\sqrt{g} \quad \log \frac{12R}{K_r}$$

$$C = \frac{1.486}{n} \quad R^{1/6}$$

Meyer-Peter and Muller -

$$T = 8 \ bd^{3/2} \ \sqrt{g\Delta} \ \left[\frac{\mu hi}{\Delta d} \ - \ 0.047\right]^{3/2}$$

$$\Delta = \frac{\rho s - \rho w}{\rho w}$$

$$\mu = \left(\frac{C}{C'}\right)^{3/2}$$

$$C' = 5.75 \ \sqrt{g} \ \log \ \frac{12R}{k_s}$$

$$C = 5.75 \ \sqrt{g} \ \log \ \frac{12R}{k_r}$$

Dominant Discharge -

$$K = \frac{mTp}{h \cdot \Delta h}$$

$$p = 1 + \left[\frac{0.25}{T/_{bd}{}^{3/2} \quad \sqrt{\Delta g}}\right]^{2/3}$$

ACKNOWLEDGMENT

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SUMMARY AND CONCLUSIONS

by

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Sediment problems encountered in water resource projects may be classified as either technical or institutional.

Technical problems involve predicting the amount and location of sediment deposits and channel degradation, bed forms and their influence on flow depths, the effect of sediment on water quality, and the influence of sediment on the esthetic value of land surrounding reservoirs. Technical problems and methods for their analyses, can be grouped according to the amount of detailed information required for a solution.

a. Problems involving navigation requirements and bank stabilization structures require detailed knowledge of sediment movement and the resulting bed forms in terms of both the time and space domains. At present, analytical methods are not adequate for complete analysis of these problems; however hydraulic model studies have been successfully utilized to develop solutions. When detailed information is needed for design purposes it is possible to pattern the design after a stable reach of river in the vicinity of the trouble spots. For example, satisfactory navigation depth and a channel which maintains itself were achieved on the Missouri River upstream from Kansas City by utilizing information from stable reaches of the river. It is, in effect, hydraulic modeling at prototype scale. However, this procedure does not reveal why a design is successful or how it can be improved.

b. In problems involving storage depletion in reservoirs and degradation downstream from dams, less detailed information is required in the time and space domains. As a result, analytical methods involving detention-time, area-increment, concentration ratio, equilibrium bed and digital modeling techniques may be used.

c. Between the above two extremes are a variety of sediment problems that require various combinations of analytical and hydraulic model studies for satisfactory analyses. Because basic relationships governing the response of an alluvial stream to its water-sediment discharge are not clearly established, considerable engineering judgment is required to properly determine and solve sediment problems. Results from both analytical and hydraulic model studies should be interpreted only by experienced investigators.

Institutional problems are the second major category of sediment problems. There is an urgent need for greater emphasis on sediment studies within the Corps. Seminar participants felt that to ensure competence in the Corps of Engineers in the area of sedimentation, the following must be done:

a. Organizations commensurate in size with the magnitude of the sediment problems, should be established in Corps offices to deal with these problems and to recruit and train new engineers to handle the complicated analyses associated with sediment studies.

b. Hydraulic models should continue to be used for basic research and the solution of specific and general Corps wide problems to gain a better understanding of the principles of sediment movement and stream behavior.

c. Prototype studies, such as those in progress on the Missouri and Mississippi Rivers, should be continued in an effort to establish the basic relationships involved in sediment transport in rivers and reservoirs.

d. New mathematic modeling techniques, involving the computer, must be developed to reproduce in more detail than presently available the hydrodynamics of streamflow. This will provide a better understanding of the important variables in sediment problems, and will directly affect the type, amount, and frequency of field data collected. This should include one-dimensional, two-dimensional and possibly three-dimensional flow models.

e. Research in Corps of Engineers offices must be coordinated with universities to insure that better techniques for solving sediment problems are forthcoming.