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Symposium 4.—Sedimentation in Reservoirs

INTRODUCTION

This symposium concerns problems directly related to sediment deposition in and above reservoirs. The papers and discussions to follow present typical problem situations encountered in the United States and describe practices used by government agencies having responsibilities for design and operation of reservoir projects. Individual papers discuss methods of estimating future conditions; surveying procedures; measuring instruments and techniques; design considerations; and measures used to alleviate certain problems. Most of the discussions pertain to problems associated with specific projects, specific types of reservoirs, or a single aspect of a reservoir sediment problem. All are concerned with one or more aspects of reservoir storage depletion and/or features associated with local deposition and erosion.

Information contained in the papers concerning reservoir surveys and related observations, reflect important improvements that have been made during the past 15 years in the development of instruments, equipment and techniques for measuring sediment. One of the most significant advances, implicit in several of the papers, has been the development of knowledge regarding optimum balance between requirements for survey facilities, survey types, and timing of resurveys necessary to accomplish the basic objectives.

Collectively, the papers show that the importance of problems varies regionally, depending upon the quantities and characteristics of materials transported by streams and according to reservoir types and purposes. However, reasonably accurate volumetric forecasts of total deposits or depletion of space within a given elevation-zone is not always indicative of the seriousness of problems that may arise. For example, storage allocated for recreation may become adversely affected for that purpose in a short period of time because of unfavorable distributions even in cases where total volumes of deposits are relatively small. It is noteworthy

that several of the papers reflect the increased attention being given to sediment distributions in investigations for project planning, design and operation.

In view of the importance attached to locations of sediment deposits, it is significant that none of the papers contain suggestions regarding approaches for rigorous analyses to predict distributions. The distributions are governed by numerous factors which include sediment quantities and characteristics; reservoir size and shape; tributary locations and relative magnitudes of sediment contributions; unpredictable sequences of extremes of hydrologic events and effects of coincidental regulation operations; and influences of unforeseen upstream developments. Because of the complexities involved, the prospects for development of precise methods for general application in predicting future distributions are not promising. However, the papers show that, for engineering purposes, considerable progress has been made in the use of empirical methods to aid in foreseeing potential problems.

As the development of survey methods and techniques has progressed with field experience, improvement of capabilities for forecasting reservoir sedimentation effects, has been coincident with a gradual accumulation of improved basic data. The Subcommittee on Sedimentation has endeavored to facilitate continued efforts in reservoir sediment investigations by arranging for exchange of pertinent reports and records prepared by various agencies. A continuing activity in this area is a cooperative arrangement whereby standardized summaries are prepared of all reliable reservoir sedimentation surveys made in the United States. These records, which are now available for more than 1,000 reservoirs of various types and sizes, are expected to be of lasting value for further studies of problems represented in the following papers.

SEDIMENT STORAGE AND MEASUREMENT IN THE UPPER COLORADO RIVER BASIN

[Paper No. 78]

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Abstract

Provision for sediment storage was a major consideration in the planning, selection, and design of the reservoirs of the Colorado River storage project in the Upper Colorado River Basin. These reservoirs are now all in final stages of planning, design, or construction, and include Flaming Gorge Reservoir on the Green River, Navajo Reservoir on the San Juan River, Blue Mesa, Morrow Point, and Crystal Reservoirs on the Gunnison River, and Lake Powell on the Colorado River. Total initial storage capacity is 33.6 million acre-feet. Estimated sediment accumulation will total 17.8 million acre-feet in 200 years. Extensive data collection facilities have been installed in the reservoir basins and in headwater and tailwater reaches to insure adequate means to accomplish periodic future measurements of deposited sediments and channel changes.

Introduction

Construction and closure of the four units of the Colorado River storage project will initiate a new stream regimen on the main stem of the Colorado River and on each of the principal tributaries in the Upper Colorado River Basin. Gone forever will be the natural flow patterns. Largely substituted for the natural hydrographs of the uncontrolled streams will be flow patterns dictated by the requirements for storage, control, and use of the water of the Upper Colorado River to meet terms of the Colorado River Compact, the Upper Colorado River Basin Compact, and the various beneficial uses to which the waters of the basin will be applied. These include irrigation, industrial and municipal, power production, flood control, recreation, and fish and wildlife uses. With the control, storage, and use of the water will come attendant changes in the movement of sediment in the main stem of the Colorado River and principal tributaries in the Upper Colorado River Basin. This paper will describe some of the activities relating to the storage and future measurement of sediment in the four units of the Colorado River storage project. Time will not permit discussion of the numerous participating projects nor more than a general description of the problems involved and the planning that has been accomplished in handling the sediment phases of the project.

Provision for sediment storage was a major

consideration in the planning, selection, and design of the principal reservoirs of the Colorado River storage project in the Upper Colorado River Basin. A basic concept adhered to was that a specified minimum active storage content should remain at the end of a 200-year period. Thus, reliable estimates were necessary to determine the storage capacity required in each of the project reservoirs to accommodate the amounts of sediment believed to be present and passing each of the damsites. Several estimates of sediment accumulation and distribution were made during various stages of planning the reservoir system as new data were collected and analyzed. These estimates have now been refined, based upon all data that are available. Further revisions will be made in the future as a result of continued measurement of sediment loads moving in the streams and observance of sediment accumulations in the reservoirs.

A brief description of each of the storage units will serve to acquaint you with the magnitude of the Colorado River storage project. Figure 1 shows the location of the reservoirs and principal geographic features of the Upper Colorado River Basin. Estimates of the amount of sediment that will be trapped in each reservoir will be presented, and the various means to measure the amount of sediment deposition and changes in stream channels in reservoir backwater and tailwater reaches will be described.

Description of Storage Units

Flaming Gorge Dam and Reservoir

Flaming Gorge Dam is located on the Green River in the northeastern corner of Utah about 40 miles north of Vernal. The dam and the lower part of the reservoir are situated in a deep gorge of the Uinta Mountains, a part of which was named Flaming Gorge by John Wesley Powell in his exploration of the Green and Colorado Rivers in 1869. The rest of the 90-mile-long reservoir extends upstream into the rolling hills and flat lands of southwestern Wyoming to the vicinity of Green River, Wyo. The reservoir will control a drainage area of 15,350 square miles and contain total storage of 3,789,000 acre-feet when full to normal water surface level of 6,040 feet m.s.l. Operation of Flaming Gorge will be geared for the maximum

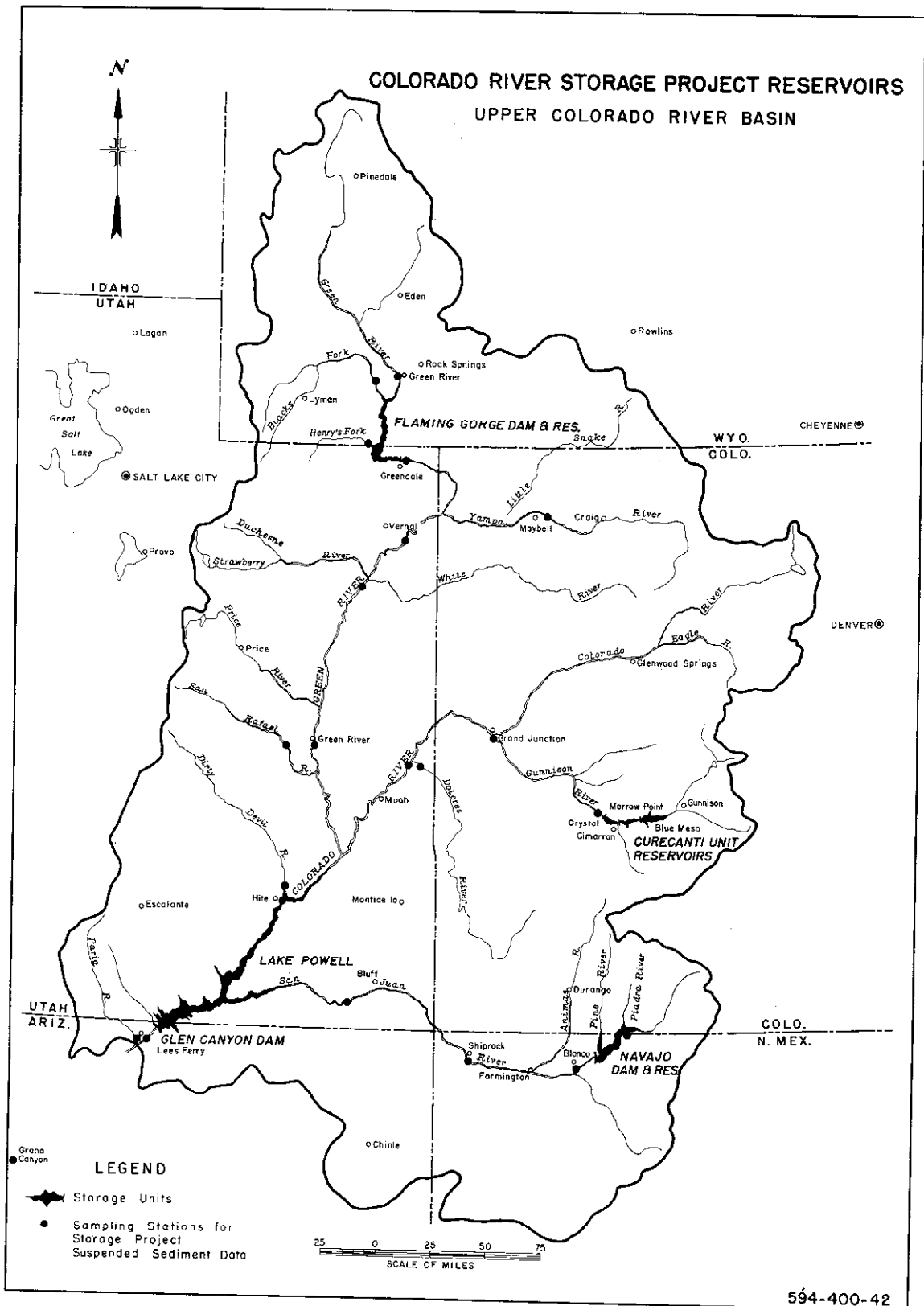


FIGURE 1.—Reservoirs and principal geographic features of the Upper Colorado River Basin.

generation of hydroelectric energy, so considerable seasonal and long term fluctuation of the water surface is expected. Flaming Gorge Dam was closed, and storage in the reservoir began on November 1, 1962.

Navajo Reservoir

Navajo Reservoir was the first storage project unit to be closed and storage initiated therein. This occurred on June 27, 1962. Navajo Dam is located on the San Juan River immediately downstream from its junction with the Pine River in northwestern New Mexico. The reservoir extends 35 miles upstream into Colorado, and the headwaters are situated in both the Piedra and San Juan Rivers at normal water surface elevation of 6,085 feet m.s.l. The drainage area is 3,190 square miles, and storage capacity totals 1,709,000 acre-feet. No powerplant was included, but the dam will form the future diversion point for the Navajo Indian irrigation project south of the San Juan River in the vicinity of Farmington, N. Mex. The bulk of the river flow will be released through the outlet works initially until major diversion demands arise for the irrigation project. This reservoir will largely regulate the San Juan River at the damsite, but sizable spills will occur in years of high runoff and several large tributaries of the San Juan River below Navajo Dam will not be regulated.

Curecanti Unit

The Curecanti unit consists at the present time of two reservoirs located on the Gunnison River downstream from Gunnison, Colo., and upstream from the mouth of the Cimarron River. Blue Mesa Reservoir, formed behind Blue Mesa Dam, is the upper and principal regulatory unit, with a storage capacity of 941,000 acre-feet at elevation 7,519.4 feet m.s.l. Morrow Point Dam, 12 miles downstream from Blue Mesa Dam, forms a 117,000-acre-foot lake in the deep, narrow Gunnison River Canyon. Crystal Dam and Reservoir, next downstream, with a storage capacity of 38,000 acre-feet, is now under active consideration. All three dams will be equipped with hydroelectric powerplants. Peaking energy will be generated at Blue Mesa and Morrow Point powerplants. Crystal Reservoir will thus be required to provide short-term regulation of releases down the Gunnison River, while Blue Mesa Reservoir will possess sufficient capacity for seasonal and long term regulation. Completion of Blue Mesa Dam is due in late 1965, and Morrow Point in 1966. No definite schedule is yet available for Crystal Dam.

Glen Canyon Dam and Lake Powell

Lake Powell, the reservoir behind Glen Canyon Dam, is the largest reservoir of the

storage project. It was appropriately named after John Wesley Powell, who, as previously mentioned, first explored the Colorado River in 1869. Glen Canyon Dam is located about 15 miles upstream from the boundary between the Upper and Lower Colorado River Basins, and the reservoir extends upriver 186 miles. In many ways, Glen Canyon Dam and Lake Powell closely resemble world famous Hoover Dam and Lake Mead. Glen Canyon Dam towers approximately 710 feet above bedrock as compared to Hoover Dam's 726 feet. Lake Powell will provide a total of 27 million acre-feet of storage at normal water surface elevation of 3,700 feet m.s.l., as compared to Lake Mead's initial capacity of 31,250,000 acre-feet.

Lake Powell is assigned the task of final regulation of the Colorado River before the stream leaves the Upper Basin. This long-term storage requirement, together with maximum power production, will cause the lake to experience cycles of filling and emptying extending over many years, similar to the pattern of operations at Lake Mead. Also, as in the case of Lake Mead, Lake Powell will be required to store a tremendous load of sediment annually. Headwaters of both reservoirs are buried in deep, narrow canyons. This is fortunate in some ways, as reservoir fluctuations will prevent formation of large exposed deltas and consequent phreatophyte problems. At the headwaters Lake Powell is divided between the main stem and its major lower tributary, the San Juan River, in some respects similar to the Colorado-Virgin River division of Lake Mead.

Construction began on Glen Canyon Dam in October 1956 and has now progressed to the point where closure of the diversion tunnels and storage of water in the reservoir are imminent. Completion is scheduled for 1964.

Sediment Storage in Reservoirs

Flaming Gorge Reservoir

Studies of suspended sediment in the Green River in the vicinity of Flaming Gorge Dam and Reservoir revealed that we could expect a mean annual accumulation of 2,040 acre-feet. These studies were initially based upon suspended sediment sampling of streams tributary to the reservoir. Sediment measuring points included the Green River at Green River, Wyo.; Blacks Fork near Green River, Wyo.; and Henrys Fork near Linwood, Utah. Further comparisons were made by utilizing downstream records on the Green River near Jensen, Ouray, and Green River, Utah, and the Yampa River at Maybell, Colo., to establish contribution rates for unmeasured areas. Final checks were completed after 2 years of suspended sediment sampling on the Green River near Greendale, Utah.

These comparative studies all confirmed the estimate of 2,040 acre-feet of annual sediment accumulation with the bulk of this amount originating in the Blacks Fork drainage and other tributaries between Green River, Wyo., and the damsite. Sediment load in the Green River at Green River, Wyo., proved to be comparatively light.

Navajo Reservoir

The San Juan River sediment load at the Navajo Dam was calculated to be 3,300 acre-feet annually. This figure is a net inflow to Navajo Reservoir after effects of the recently authorized San Juan-Chama transmountain diversion from the headwaters of the San Juan River were accounted for. Studies were based upon several years' record of the San Juan River at Blanco, N. Mex., and by comparison with other long term records on the river. The dam lies upstream from the heaviest sediment-producing areas in the San Juan River drainage.

Curecanti Unit

The reservoirs of this unit are located on the exceptionally clean, sedimentwise, upper Gunnison River. Estimates indicate that less than 340 acre-feet of sediment would accumulate annually in Blue Mesa Reservoir and about 28 acre-feet in Morrow Point. The proposed Crystal Reservoir will receive and store approximately 38 acre-feet annually, mostly from the Cimarron River drainage.

Lake Powell

Far and away the largest sediment load affecting the storage units will be carried by the Colorado and San Juan Rivers into and stored in Lake Powell, the principal regulatory reservoir of the Upper Colorado River Basin. Lake Powell is situated below the great sediment-producing areas in Colorado, Utah, New Mexico, and Arizona, tributary to the Upper Colorado River Basin. Fortunately, extensive suspended sediment records, some of which date back to 1925, are available for the Green, San Juan, and Colorado Rivers above the reservoir and at Lees Ferry and Grand Canyon below the dam. After careful consideration of the suspended sediment and water records and the possible effect of major authorized upstream storage units, we estimated that an average of 104,000 acre-feet of sediment could be expected to flow into Lake Powell annually. Compaction of the sediments in a 100-year period would reduce the 104,000 acre-feet to a mean annual volume of 85,400 acre-feet. Thus, the storage requirement for sediment was calculated to be 8,540,000 acre-feet for a 100-year period. Further compaction results in a 16,800,000-acre-foot requirement for a 200-year period.

Possible long term effects of participating project developments were not considered. This figure also takes full cognizance of and is partially dependent upon records of the main Colorado River back to 1925, although records since 1943 indicate considerably less sediment was carried annually in the Colorado River since 1943 than the mean annual load previous to that year. Consequently, we feel this amount is an adequate reflection of any likely long-term sediment-producing condition on the Colorado River. The actual measured accumulation in Lake Mead for the period 1935-48, inclusive, also offers a check on our calculations.

Special Problems and Plans for Future Measurement of Sedimentation

Flaming Gorge

Storage of the relatively small amount of annual sediment inflow offers no particular problems for Flaming Gorge Reservoir, as the 200-year sediment load of 398,000 acre-feet is expected to occupy only about 10 percent of the total reservoir capacity. A special topographic map of the reservoir basin has been completed that will provide basic reference data for any needed remeasurement in the reservoir basin. Bedload and delta formation in the headwaters of the reservoir on the Green River and on Blacks Fork were investigated to predict the possible backwater effects on improvements near the reservoir.

The city of Green River, Wyo., lies on the bank of the Green River, and the Union Pacific Railroad and Highway 30 cross the Green River near the city about 7 miles upstream from the normal water surface of Flaming Gorge Reservoir. An extensive network of 27 permanent ranges was installed beginning at the railroad bridge and extending downstream to where the last six ranges lie in the reservoir basin. These ranges were surveyed to provide data for backwater studies, with an expected sediment delta in place and for future measurement of delta formations and channel changes in the headwaters of the reservoir and in the channel reaches upstream to Green River. Studies showed no backwater effects from the reservoir would extend upriver to threaten any of the improvements previously mentioned.

A short section of Wyoming State Highway 530 and the highway bridge across Blacks Fork on the west side of Flaming Gorge Reservoir were required to be raised because of water backing up Blacks Fork from the reservoir. The present location offered some advantages for the crossing rather than crossing above the reservoir line. Blacks Fork carries about 750 acre-feet of sediment per year into Flaming

Gorge, and a considerable portion of this is expected to form a delta in the Blacks Fork channel where little storage space is available. Consequently, studies were completed to establish minimum clearance for the highway bridge under conditions of delta formation and expected flood flows. Fifteen sediment ranges were surveyed to furnish required data. All ranges were permanently monumented.

Navajo Reservoir

Construction of the Navajo Dam and Reservoir required the relocation of the Denver & Rio Grande Western's narrow gage railroad and State and county roads near the head of the reservoir along the San Juan and across the Piedra Rivers. Sediment delta and backwater studies were completed after additional suspended sediment records were gathered and permanent ranges installed. The data were then utilized to select locations and elevations of roadbeds, including the bridge crossings of the Piedra River.

Expected deposition of sediment in Navajo Reservoir will total about 532,000 acre-feet in a 200-year period and will occupy about 31 percent of the initial capacity. This estimate indicated the need for accurate periodic remeasurement of the reservoir basin to adjust area-capacity data and operation schedules.

To best fill this need, a topographic map with 10-foot contours was obtained and a comprehensive network of permanent sediment ranges was designed and installed throughout the reservoir basin and headwater delta reaches. This network consists of 82 permanently monumented ranges at about 1-mile intervals linked together by a common survey and based upon national systems of horizontal and vertical control. These ranges will provide adequate basic data for reservoir resurveys at any time required by project operations.

Regulation of the San Juan River and relief of its sediment load in Navajo Reservoir may cause changes in the channel below the dam. Many side canyons and washes draining to the San Juan are subjected to flash floods and occasionally transport and deposit of heavy loads of sediment in the San Juan River channel. Thus, the San Juan River channel could experience degradation in some sections of the channel and aggradation in other parts of the reach from Navajo Dam to Shiprock, N. Mex. To measure these channel changes, a system of river ranges is now being installed downstream to Shiprock, with the bulk of the ranges occurring between the dam and Blanco. Many of these ranges will be below the mouths of tributary canyons, where changes in channel conditions seem most likely. Additional cross section data are available at the several Geological

Survey stations along the river, and the State of New Mexico has expressed interest in surveying additional ranges or obtaining new aerial photographic coverage of the river channel.

Curecanti Unit

Authorizing legislation for the Colorado River storage project specified that a storage reservoir of not less than 940,000 acre-feet should be constructed on the Gunnison River, or such greater capacity as could be obtained with a water surface at 7,520 feet m.s.l. at the Blue Mesa site. A water surface elevation of 7,519.4 feet m.s.l. was needed to provide 941,000 acre-feet of storage. Prospective delta formation was believed to be minor for Blue Mesa Reservoir, but backwater studies with delta formations were made to determine if any encroachment would take place. Calculations showed that reservoir and headwater channel clearing of brush and trees would offset any rise in water surface elevations caused by this minor delta formation. Sediment ranges were established by permanent markers.

The Cimarron River enters the Gunnison in the reach common to the tailwater of Morrow Point and headwater of the prospective Crystal Reservoir. Studies have now been completed on delta formation in this reach to furnish needed design data.

Expected reservoir sediment depositions will not cause significant deleterious effect in Blue Mesa or Morrow Point Reservoirs, but sediment will occupy upward of 20 percent of the total storage in Crystal Reservoir in a 200-year period.

Lake Powell

The accumulation of sediment expected in Lake Powell, as described previously, constitutes a noteworthy factor in the operation and management of the reservoir. The 200-year sediment accumulation of 16,800,000 acre-feet will eventually occupy about 62 percent of the initial capacity. Therefore, careful consideration was given to provide adequate basic data before the reservoir filled that would permit periodic measurement of the accumulated sediments and also be used for research purposes as the need arises for additional studies. The rugged and inaccessible walls of Glen Canyon of the Colorado River soon convinced us that the conventional method of field surveying and installation of sediment ranges in Lake Powell would not be practical or economical. Consideration was then given to establishment of sufficient basic vertical and horizontal control that would permit construction of sediment ranges or strips of topography across the reservoir by photogrammetric means. An alternate to this latter scheme was a proposal to remap the en-

tire reservoir basin at a large scale and small contour interval.

The remapping plan was finally adopted, as its estimated cost was approximately the same as for providing sediment ranges photogrammetrically. The complete map offered much more freedom for remeasurement for sediment purposes and also furnished a large-scale map for an accurate initial determination of area and capacity data and for many other uses associated with the development and management of the reservoir. I have indicated the topographic coverage was a "remapping" procedure. Modern map coverage was available for Lake Powell but only at a scale of approximately 1 inch equals 1 mile and with a contour interval of 80 feet. Specifications for the new map called for a scale of 1 inch equals 400 feet and contour interval of 10 feet.

The cooperative efforts of three agencies of the Federal Government and several private firms produced a map of high standard that covered the entire reservoir to 3,750 feet m.s.l., some 50 feet above the normal water surface. The excellent vertical photographic coverage of the natural reservoir basin is also a fine by-product. This mapping required unique applications of vertical control procedures and called for large scale use of helicopter transportation to establish control surveys. The complete mapping job is described in the article by Gessel and Rutledge.¹

Extensive first-order level lines joined to the national leveling network were also established throughout the reservoir basin, to make available precise data for measurement of sediment deposition in the future and to allow installation of ranges when operating personnel will be able to work from the lake surface. These high-order level lines will also allow detection and measurement of possible displacement of the earth's crust in the reservoir basin due to the concentration of approximately 37 billion tons of water and sediment.

Predictions of the behavior of the Colorado River channel below Glen Canyon Dam were needed to determine design data for the dam and powerplant. It appeared that the Paria rifle, some 15 miles downstream, offered the closest stable downstream control section, so 21 ranges were surveyed in the reach between the dam and the mouth of the Paria River. These ranges have been resurveyed several times and revealed some heavy degradation in the stream channel. Other resurveys will be made as required.

Long range planning for recreational development on Lake Powell required consideration of sediment aspects. The National Park Service was interested in establishing a major recreational development near the present Hite Ferry crossing in the upper reaches of Lake Powell. After consultation and study by the Bureau of Reclamation, it was shown that an installation at this location would have a comparatively short life due to sediment delta encroachment. The Bureau study made for this purpose drew heavily from data and experience of Lake Mead, where the sediment delta has now progressed downstream and largely engulfed the Pierce basin.

A resurvey of the delta in Lake Mead in early 1961 revealed a topset slope of 1 foot per mile. This topset slope data, together with the predicted amount of sediment inflow from the Colorado River arm and the cross sectional data available from the new topographic map resulted in an estimate of 20 to 25 years before the Hite basin would be engulfed and rendered hazardous for recreational purposes.

Another location accessible by road near the mouth of Red Creek about 12 miles downstream would likely survive for a 40- to 50-year period. We understand the Park Service is considering this site. Use of an electronic data computer greatly speeded the several trial studies required, as additional studies were computed rapidly after initial data preparation was complete. Guidance was also given to solve a similar problem on the San Juan arm of the reservoir. Here again, it was revealed that the most readily accessible area on the San Juan part of Lake Powell will be affected by sediment shortly after storage in the reservoir begins.

Closure of the coffer dam and diversion of the Colorado River in February 1959 has caused significant amounts of sediment to deposit in Lake Powell during construction of Glen Canyon Dam. This is evidenced by the greatly reduced measurements of suspended sediment in the Colorado River at Lees Ferry and the visible deposits in the reservoir basin. Quantitative measurements were not taken in the reservoir basin, but, from study of flow and sediment records at Lees Ferry, we estimate that deposits in the order of 60,000 acre-feet accrued in the 1959-61 water years. Comparison of records on the tributary streams above the reservoir basin with records for Lees Ferry in 1960 also showed deposition occurred. The total suspended load for 1960 at the three principal stations of San Juan River near Bluff, Colorado River near Cisco, and Green River at Green River, Utah, was 116 percent of the load at Lees Ferry. The highest percentage for

¹ GESSEL, C. D., and RUTLEDGE, D. H. LARGE SCALE MAPPING OF LAKE POWELL. Amer. Soc. Civil Engin., Jour. of the Surveying and Mapping Div., Paper 3318, 17 pp. 1962.

previous years was 79 in 1954, while the average was 62 percent for the 1943-58 period.

Discussion

To summarize, it is apparent that in a space of less than a decade the stream regimen in much of the Upper Colorado River will be significantly changed as the reservoirs of the Colorado River storage project will regulate the principal tributaries and store great quantities of sediment. Lake Powell, by far the largest and most important reservoir, will bear the greatest burden. It will trap 16,800,000 acre-feet of sediment in 200 years. Navajo Reservoir will accumulate 532,000 acre-feet in the same period, while the sediment loads in Flaming

Gorge and the Curecanti unit reservoirs are relatively minor when judged against the capacities and functions of these reservoirs. Overall, the storage project reservoirs will be called upon to store about 17.8 million acre-feet of sediment in 200 years, or 53 percent of their total initial capacity (table 1). Many studies have been completed to solve special problems associated with sediment. Data collection facilities have been installed that will provide information for future operational and research purposes.

We intend to watch the sediment phases of this project with great interest in the coming years and accomplish necessary data gathering and additional studies as the needs arise.

TABLE 1.—*Estimated sediment accumulation, Colorado River storage project reservoirs, 1963*

Reservoir	River	Total capacity	Annual sediment inflow	100-year sediment accumulation ¹	200-year sediment accumulation ¹	200-year condition, percentage of initial capacity ¹
		<i>Acre-feet</i>	<i>Acre-feet</i>	<i>Acre-feet</i>	<i>Acre-feet</i>	
Flaming Gorge	Green	3,789,000	2,040	201,000	398,000	10.5
Navajo	San Juan	1,709,000	3,300	276,000	532,000	31.1
Blue Mesa	Gunnison	941,000	340	33,300	66,300	7.0
Morrow Point	do.	117,000	28	2,700	5,400	4.6
Crystal	do.	38,000	38	3,800	7,500	19.7
Lake Powell	Colorado	27,000,000	104,000	8,540,000	16,800,000	62.2
Totals		33,594,000		9,056,800	17,809,200	53.0

¹ 100-year and 200-year figures are compacted sediment amounts.

SEDIMENTATION IN WALLA WALLA ARM OF McNARY RESERVOIR AND ITS EFFECT ON RECREATIONAL DEVELOPMENT

[Paper No. 79]

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Synopsis

The preliminary master plan for use of the reservoir created by McNary Dam on the Columbia River provided for a recreational park to be developed by the State of Washington where the Walla Walla River enters the reservoir. Subsequent investigations led to the abandonment of the site as a State park because of predicted deposition of sediment carried by the Walla Walla River. Sediment ranges were established and five surveys have been made since impoundment of water in 1953. Results of these surveys through 1961 show an average deposition rate of 860 acre-feet per year. This indicates the water storage area below normal pool, elevation 340 feet m.s.l., in the Walla Walla arm will be essentially filled with sediment by the fall of 1965, except for the river channel. The deposition rate is equivalent to

0.49 acre-foot per year per square mile of drainage area. Plans have been investigated to protect portions of the water area for boating and swimming by construction of dikes and intermittent dredging, but as yet these have not proved feasible. As deposition progresses it will enter into the main reservoir and may adversely affect navigation in the area being developed by Walla Walla Port district. Sedimentation also is an important factor in planning other recreational developments along the Columbia and Snake Rivers.

Introduction

This paper presents the results of a sediment study of the Walla Walla arm of McNary Reservoir and the effect of this sedimentation on recreational development.

The initial plans for recreational development of the reservoir (Wallula Lake) created

by McNary Dam provided for a recreational park to be developed in the area where Walla Walla River enters the reservoir. The Walla Walla River drains a large agricultural area, much of which is devoted to production of wheat and peas. Heavy rains or rapid snow-melt and thawing of frozen ground in spring cause large quantities of sediment to enter streams tributary to Walla Walla River, and much of this sediment is carried down to the mouth of the river.

Based on the limited data available before impoundment, it was predicted that most of the water area in the Walla Walla arm would fill with sediment in a relatively short time. Plans for the State park development therefore were abandoned. However, it was desired that provisions be made for as much recreational use of the area as practicable. Sediment ranges were established and have been periodically surveyed since impoundment of water in 1953 to determine the distribution and rate of deposition. The results of surveys made to date are presented, including preliminary information on the relationship of sediment accumulation to streamflow and drainage area. Also, general information is presented on the influence of sedimentation in the planning of recreational developments at some other locations in the Walla Walla District of the Corps of Engineers.

Description of Area

McNary Dam and Reservoir

McNary Dam is located 292 miles above the mouth of the Columbia River, 161 miles east of Portland, Oreg., and 2½ miles upstream from Umatilla, Oreg. (fig. 1). About one-half of the dam is located in Washington and one-half in Oregon. The dam is constructed of concrete and earth and was completed in 1953. Overall length of the dam is 7,365 feet. Height above streambed is 100 feet, and the normal pool elevation of the dam is 340 feet above mean sea level. Backwater effects extend about 62 miles upstream on the Columbia River, 10 miles upstream on the Snake River, 9 miles up the Walla Walla River, and 6 miles up the Yakima River. The greatest width of the reservoir is about 2 miles and is near the mouth of the Walla Walla River which enters the reservoir on the left side about 25 miles upstream from McNary Dam.

McNary Lock and Dam is one multiple-purpose unit of the main control plan for the comprehensive development of the Columbia River and tributaries. Its primary uses are power and navigation with secondary benefits from flood control, irrigation, recreation, industrial site development, and other miscellaneous benefits. Pondage of the reservoir may be used on a regular schedule for peak power production,

with drawdowns of 2 or 3 feet in the ordinary operation of the project and a maximum draw-down of 5 feet. Since impoundment of water in 1953 the maximum fluctuation in water surface elevation at the mouth of Walla Walla River has been about 5 feet. Maximum fluctuations of reservoir elevations at the mouth of the Walla Walla River are shown in figure 2.

Walla Walla River

Basin description

The Walla Walla River rises in the rugged Blue Mountains near Walla Walla, Wash., and flows about 75 miles westerly before emptying into the reservoir (fig. 3). The Touchet River is the principal tributary and rises in another part of the Blue Mountains near Dayton, Wash. It flows southwesterly for 75 miles before joining the Walla Walla River near Touchet, Wash. The Walla Walla River drains an area of about 1,700 square miles located in northeastern Oregon and southeastern Washington. The watershed from the reservoir to the mountains is predominantly an agricultural area. The hill and valley plain lands are extensively utilized for agricultural purposes, primarily for the production of wheat, barley, and peas. Land used for grain and peas is the primary source of sediment. The practice of cultivating much of this grainland in the fall increases the susceptibility to erosion. The higher slopes of the Blue Mountains are timbered and produce little sediment.

The climate is characterized by moderate temperatures and a large annual range in temperatures, with light to moderate precipitation. Average normal temperatures at Walla Walla range from 33° F. in January to 74° in July. Extremes have ranged from minus 29° to 110°. Mean annual precipitation ranges from about 8 inches near the mouth of the Walla Walla River to about 45 inches in the Blue Mountains, and averages 21 inches over the basin. Summer precipitation is usually very light. High rainfall intensities are infrequent; the highest observed was 2.4 inches in 40 minutes.

The Walla Walla River basin is underlain almost entirely by Columbia River basalt flows, which have been somewhat folded and locally faulted. Occasional sedimentary interbeds are found, and the basalt is overlain locally by gravels and generally by loess, with a minimum thickness of 10 to 15 feet. Alluvial fans are present near the mountains and are comprised of relatively homogeneous sediments consisting principally of subrounded pebbles and cobbles often interbedded with sand containing some silt. These deposits are pervious and become partially "silted in," but there is always considerable downward percolation to the water

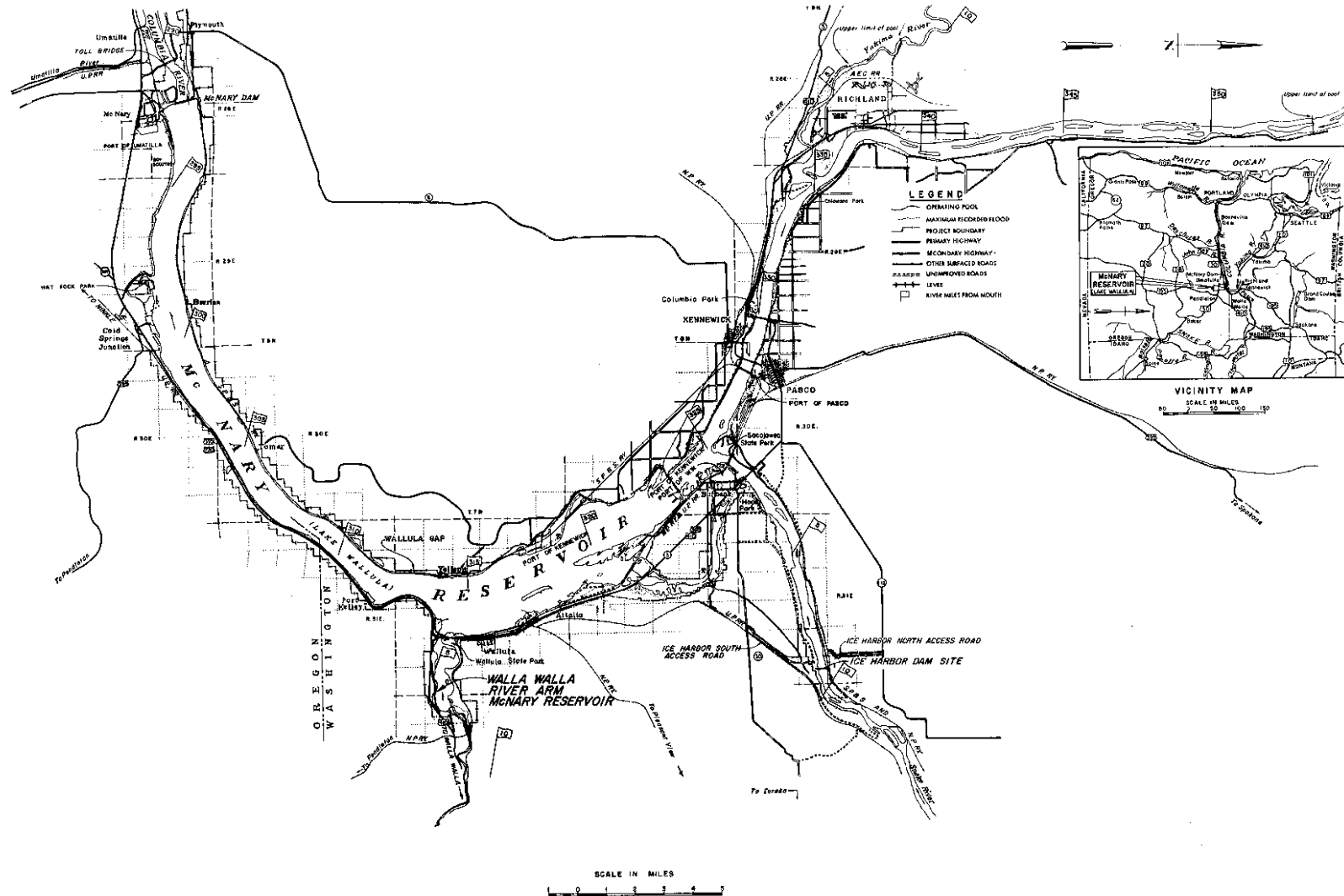


FIGURE 1. — Columbia River, Oregon and Washington.

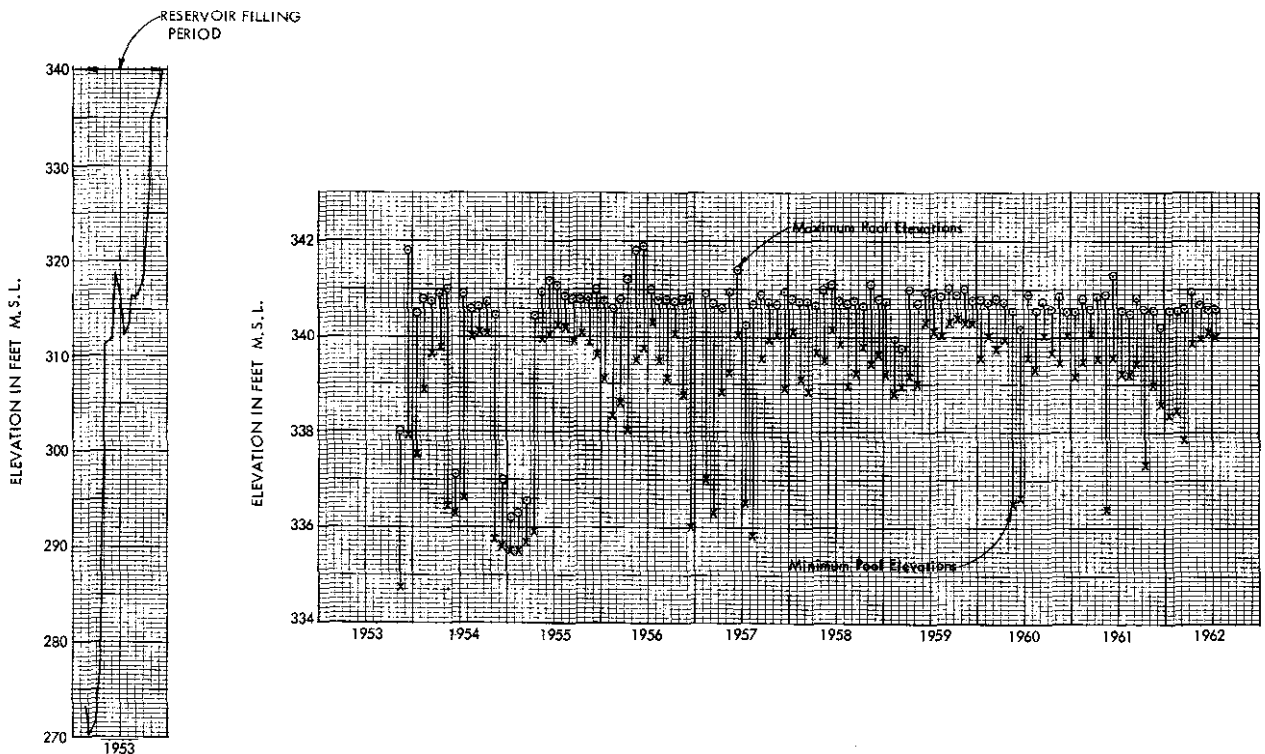


FIGURE 2. — Monthly reservoir elevations, mouth of Walla Walla River.

table, which underlies the surface at a depth of about 30 feet. Stream aggradation is continuing throughout the fan areas.

Streamflow

In general, the streamflow characteristics of the Walla Walla River basin are low flows from July through November, with occasional high flows in December, January, and February. On the average, the highest annual flows occur during the period March through June. Major floods in the area result from rainfall or a combination of rainfall and snowmelt on saturated or frozen ground conditions. Rapid thaw of the top layer of frozen ground during these occurrences results in heavy erosion of cultivated land and high concentration of sediment in the Walla Walla River. When snowmelt augments rainfall runoff, the floods have both high peaks and large volumes.

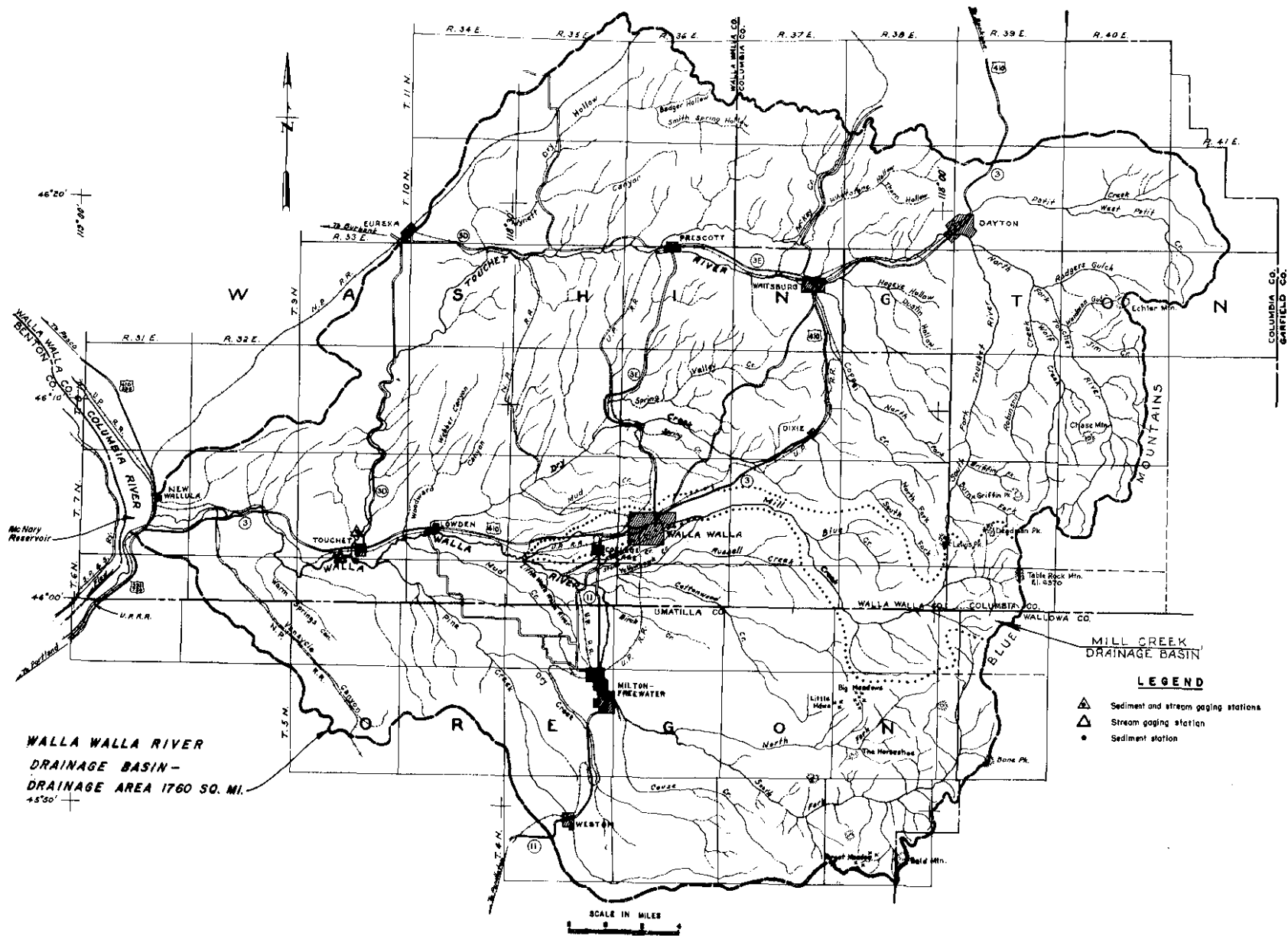
Walla Walla River below Touchet River produces an estimated average annual runoff of 440,000 acre-feet, or 5 inches over the 1,660-square-mile drainage area, based on brief records at this location and longer records in headwater areas and other streams. Individual yearly runoffs vary with annual precipitation received, and based on above records very likely have ranged from about 60

to 180 percent of the mean. Streamflows vary greatly during the year. The long-term mean flow is about 600 c.f.s., but in the brief period of record at the gaging station below Touchet River flows of 16,300 c.f.s. and 11 c.f.s. have been recorded. Flows have a seasonal pattern, with consistently low flow during the summer and fall months and during cold weather in the winter months. Table 1 is a summary of discharge data for the period since installation of stream gage in 1953.

TABLE 1.—Summary of runoff data for Walla Walla River near Touchet, Wash., 1953-61

[Drainage area, 1,600 square miles]

Month	Mean	Monthly runoff		Daily runoff	
		Max.	Min.	Max.	Min.
	C.f.s.	C.f.s.	C.f.s.	C.f.s.	C.f.s.
January.....	975	1,888	334	6,040	200
February.....	1,318	2,096	593	4,860	250
March.....	1,211	1,843	518	3,730	282
April.....	1,290	2,165	932	5,610	412
May.....	852	1,217	298	2,770	186
June.....	241	471	142	1,680	8.8
July.....	27	46	7	88	2.3
August.....	15	34	4	78	2.1
September.....	50	181	14	850	5.8
October.....	126	336	63	540	37
November.....	305	531	124	1,510	72
December.....	785	1,807	287	5,950	180



Moderately high flows occur for several weeks in the late winter and spring as a result of snowmelt, and the larger part of the normal year's runoff volume is from this latter source. The stream slopes vary from 100 feet per mile in the upper reaches of the stream to 35 feet per mile in the lower reaches.

Sediment Sampling

Suspended sediment samples were obtained at two stations in the Walla Walla River basin on a regular basis during the period 1951 through 1953. These stations were located respectively on the Walla Walla River and the Touchet River immediately above their confluence (fig. 3). The suspended samples were obtained with a D-43 sampler. Concentration of sediment varied from 0 to 45,000 p.p.m. on Walla Walla River and from 0 to 82,000 p.p.m. on Touchet River. Graphs of the flow of the streams and concentration of sediment are shown in figures 4 and 5.

The somewhat erratic nature of the relation between discharge and sediment content is indicated by comparison of the two graphs. This characteristic is explained by the variation in soil and runoff conditions that exist during the year. Also, it will be noted from figures 4 and 5 that most of the heavy concentration of sediment occurs during relatively short periods during the winter and spring. These represent periods of heavy runoff from the agricultural area caused by rains, rapid snowmelt, rapid thaw of frozen ground, or a combination of these factors. The gradual snowmelt runoff from the higher elevation area does not carry much sediment.

No measurement was made of the movement of bedload other than could be measured by D-43 sampler. However, observations at the bed at low water and analyses of suspended, bed, and deposited material (figs. 6, 7, 8) indicated that the bedload was relatively small, and therefore it was estimated at 10 percent of the suspended load. The total average annual sediment load during this 3-year period was estimated at 1,540 acre-feet, as shown in table 2.

The average annual runoff for this 3-year period is only slightly below the long-term aver-

age. Consequently, the average annual sediment load of 1,540 acre-feet may be considered fairly representative of that for a long-term period.

Analyses were made of selected samples of the suspended and bed materials at the above stations and at accessible locations on the sediment delta between ranges 4 and 5. Typical gradation curves are shown in figures 6, 7 and 8. Average of gradation curve values show about 80 percent of the sediment to be silt and clay.

Sediment Surveys

As a basis for measurement of the rate and distribution of sediment deposits, 15 sediment ranges were established during 1951 prior to impoundment of water in the reservoir. During 1958, six additional ranges were established downstream of range 1 to extend the measurements out into the main body of the reservoir. The locations of 9 of the 15 original ranges are shown in figures 9 and 10. All ranges were marked with concrete monuments to establish a permanent system for periodic surveys.

Five sediment surveys have been made since establishment of the 15 ranges. Of these, 3 surveys were completed for the 15 original ranges and the 6 ranges established in 1958. The other two surveys have been confined to the ranges in the area receiving major deposits. Soundings were made with a fathometer. Targets and transit were used for keeping the moving boat on line. The ranges and cut-in points located to give suitable cut-in angles are plotted on plane table sheets. Check points along the continuous range profiles are made by fixes on fathometer charts that correspond with alidade intersections of the range line on plane table sheet coordinated by portable radio. Figures 11 and 12 show survey crew and equipment. The 4-wheel drive vehicle, boat trailer, 16-foot jet-propelled sounding boat, and crew are shown in the process of launching in figure 11. A close-up view of the fathometer is also shown. Figure 12 shows plane table being oriented over cut-in point and boat proceeding to range location to start sounding operations.

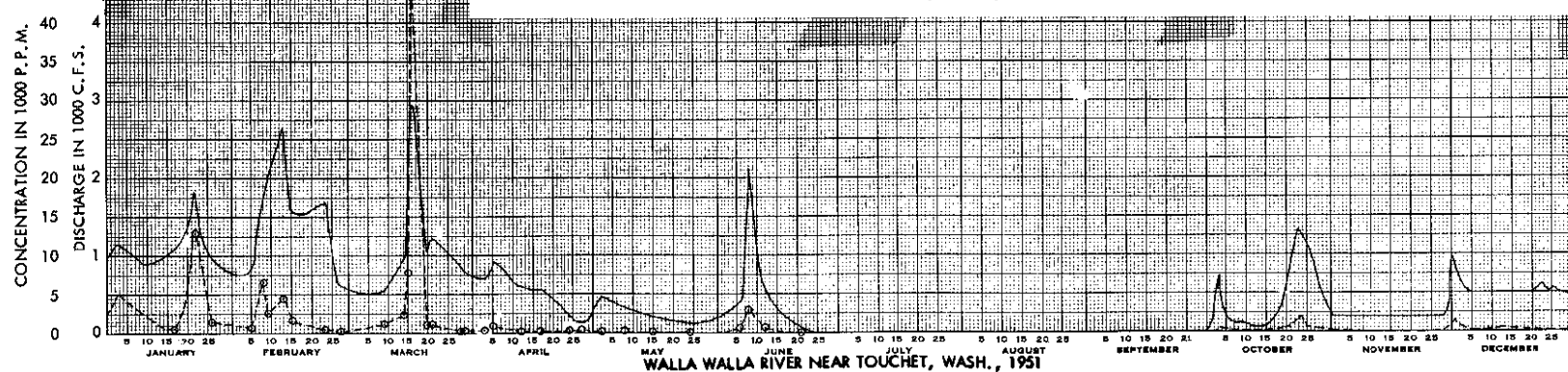
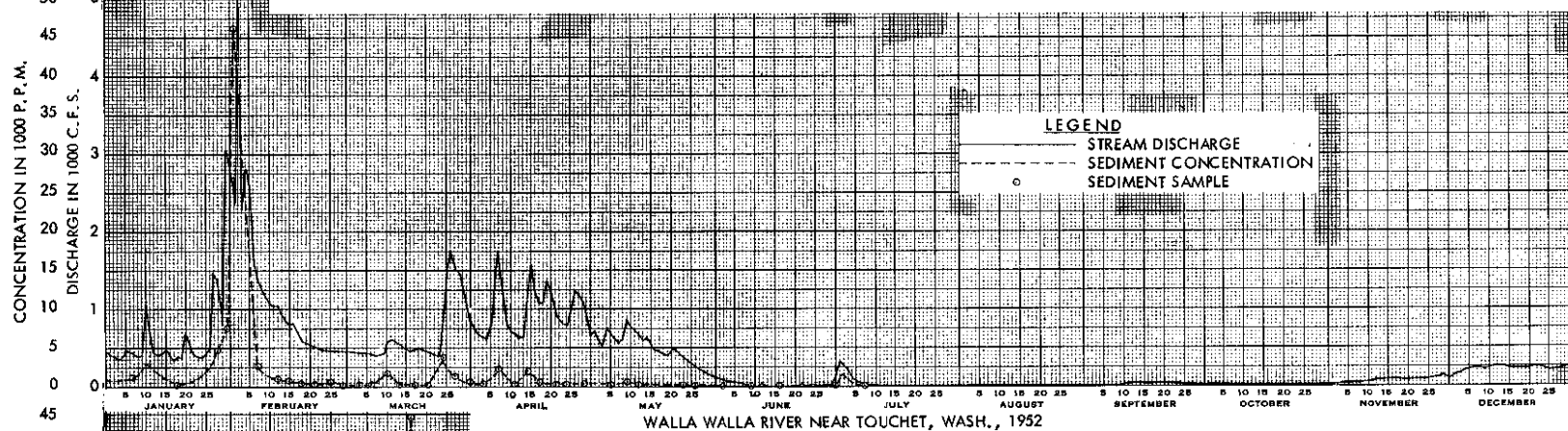
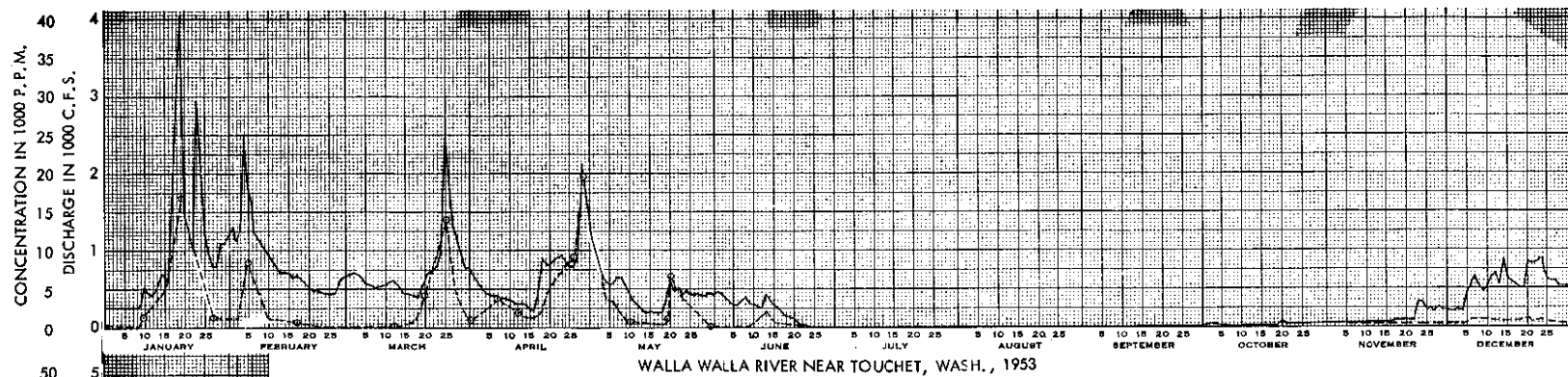
Sediment Deposits

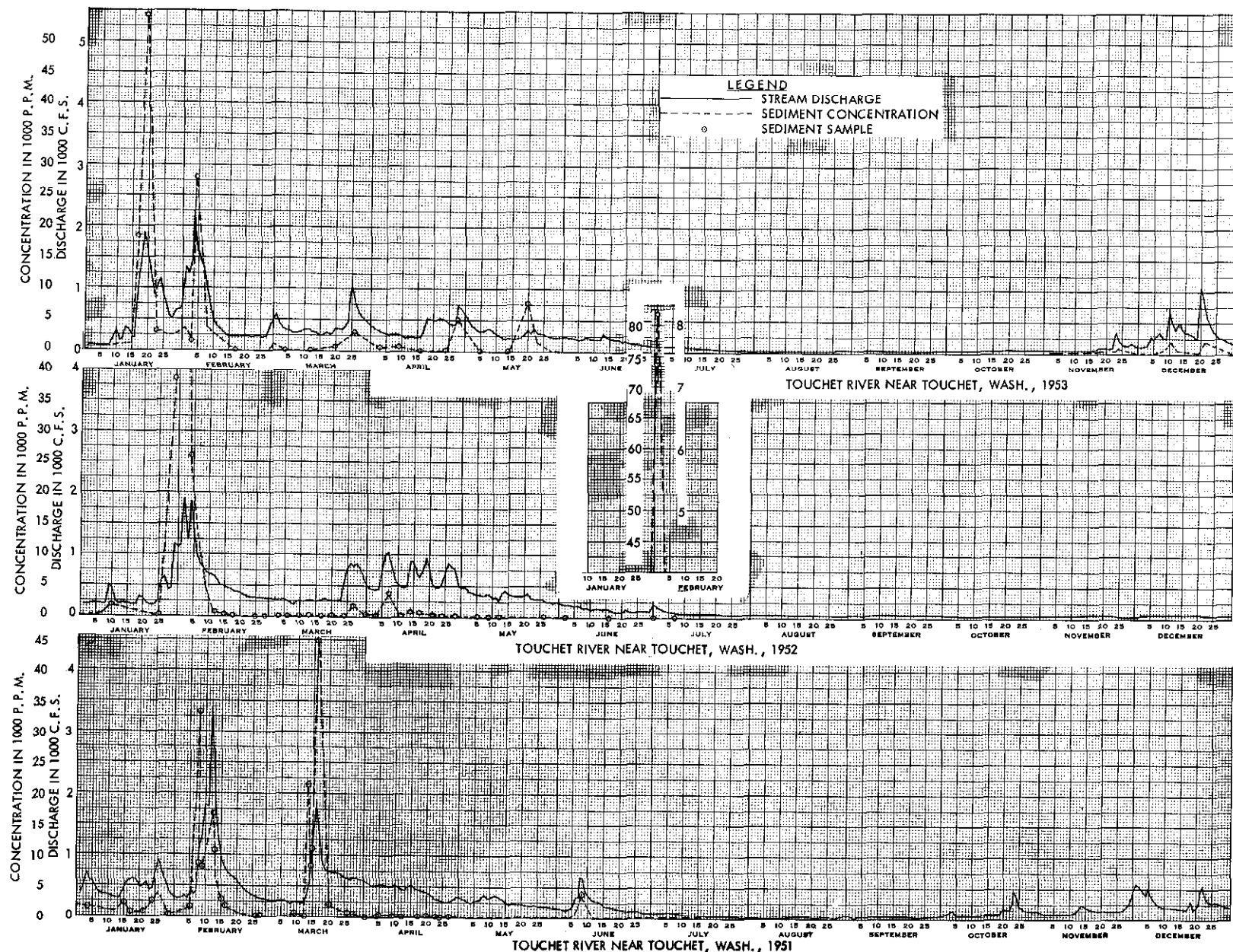
Analyses of sediment range surveys show rapid filling of the water area in the Walla Walla arm of the reservoir. As would be expected, heaviest deposits started at the head end of the reservoir arm and have progressed each year toward the main reservoir. The progressive deposition at each range is shown in figures 13, 14, and 15. No appreciable deposition has occurred downstream from range 1 or upstream from range 9. Depths of deposit

TABLE 2.—*Annual runoff and sediment load, Walla Walla River, 1951-53*

Year	Runoff	Sediment load ¹
	<i>Acre-feet</i>	<i>Acre-feet</i>
1951	360,000	1,310
1952	419,200	1,880
1953	462,100	1,420
Average	413,800	1,540

¹From sediment samples; includes 10 percent bedload.





are as great as 28 feet over the original channel and 18 feet over the original valley floor, as shown on profiles for ranges 4 and 3, respectively. A longitudinal sediment delta profile is shown in figure 16. A large proportion of deposition during the year 1961-62 occurred between ranges 2 and 3 and therefore does not show on the range profiles. The areal extent of deposition as of May 1961 is indicated pictorially by the aerial mosaic view in figures 9 and 10. Approximately two-thirds of the original water surface area upstream of range 1 has been covered with sediment to near normal pool level. Heavy growth of willows and cattails now covers much of this area. This vegetal growth is expected to increase annually and induce further sedimentation during major floods.

The total deposition between date of impoundment, 1953, and August 1961, is computed to be 6,040 acre-feet. The rate of deposition between surveys is shown in table 3.

Data for the 1962 survey were not received in time for analysis and inclusion in this table. The average rate of deposition for the 8-year

TABLE 3.—Average annual runoff and sediment accumulation, Walla Walla River

Period	Runoff	Sediment deposition
	<i>Acre-feet</i>	<i>Acre-feet</i>
1953-56.....	420,400	860
1953-58.....	441,700	850
1953-60.....	435,200	740
1953-61.....	434,900	860

period 1953-61 is 860 acre-feet. This is equivalent to a rate per year per square mile of drainage area of 0.49 acre-foot. Table 3 also shows a ratio between annual deposition and volume of runoff in acre-feet of about 1 to 500. Comparing the above rate of deposition with the average annual sediment discharge of 1,540 acre-feet estimated from suspended samples indicates that 56 percent of the total sediment load of the Walla Walla River has been deposited in the Walla Walla River arm of the reservoir.

Analyses of deposited sediment were made from samples obtained at four locations (table 4 and figure 8) and show somewhat "soupy"

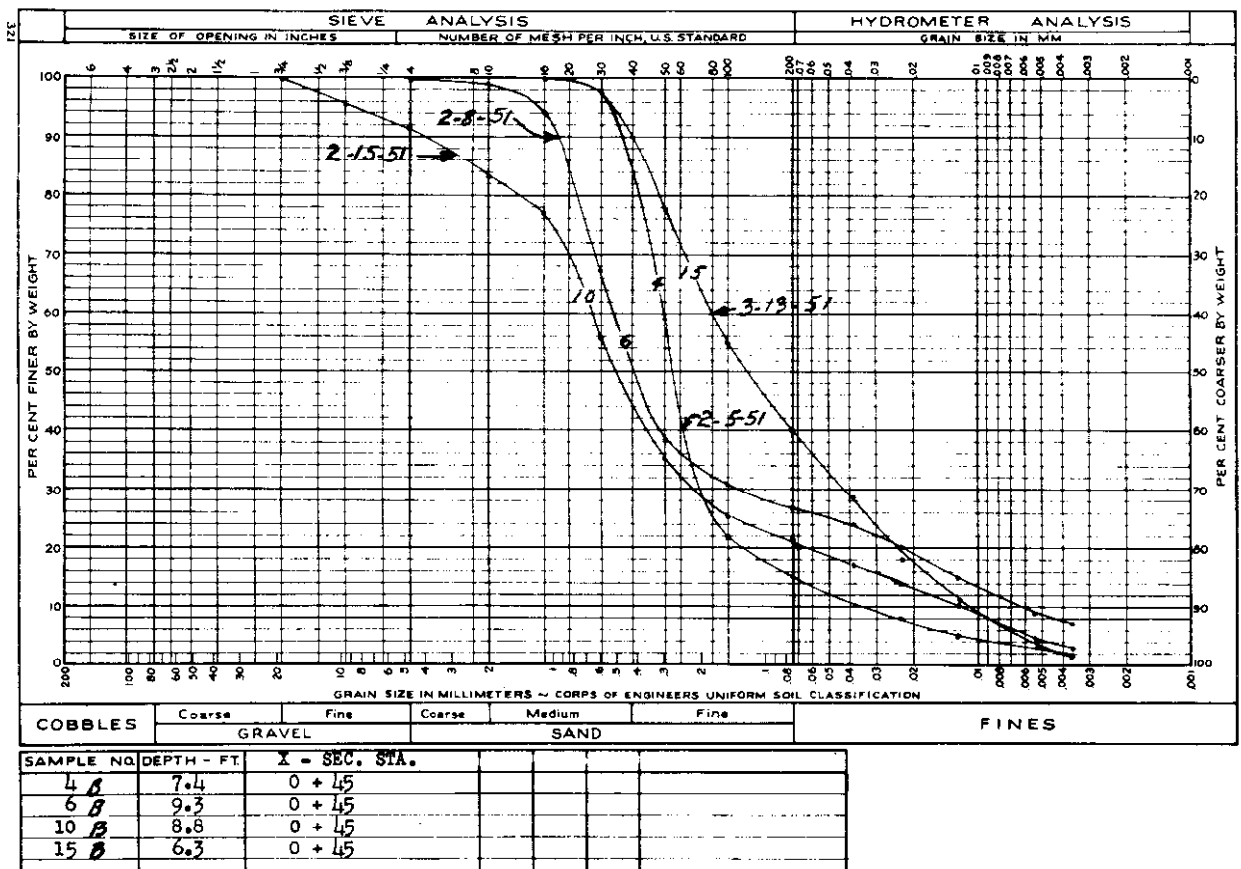


FIGURE 6. — Gradation curves, bed samples, Walla Walla River, Touchet, Wash.

TABLE 4.—Analyses of delta samples collected in 1960, Walla Walla River arm, McNary Reservoir

Hole No.	Sample No.	Sample run ¹	Approximate depth of sample	Net weight of material	Moisture content	Wet density	Dry density	Specific gravity
			Feet	Grams	Percent	Lb./cu. ft.	Lb./cu. ft.	
P-1.....	1	1.0-4.0	2.5-3.1	121.3	33.5	112.7	84.3	
	1	2.0-4.0	2.5-3.0	111.3	34.8	103.4	76.7	2.53
	2	2.0-4.0	3.0-3.5	122.1	34.8	113.4	84.2	
P-2.....	3	2.0-4.0	3.5-4.0	115.2	36.7	107.0	78.4	
	4	4.5-6.0	4.5-5.0	² 52.4	36.3			
	5	4.5-6.0	5.0-5.5	119.0	39.6	110.6	79.1	2.65
	6	4.5-6.0	5.5-6.0	118.3	38.5	109.9	79.4	
	1	1.6-4.0	1.6-2.1	² 51.3	31.6			
	2	1.6-4.0	2.1-2.6	116.0	39.3	107.8	77.3	
	3	1.6-4.0	2.6-3.1	111.8	43.2	103.9	72.4	2.55
P-3.....	4	4.0-8.0	4.0-4.5	² 100.6	59.7			
	5	4.0-8.0	4.5-5.0	² 99.2	50.7			
	6	4.0-8.0	6.5-7.0	108.0	44.5	100.3	69.4	2.52
	7	4.0-8.0	7.0-7.5	117.0	44.0	108.7	75.5	
	8	4.0-8.0	7.5-8.0	120.9	43.3	112.3	78.3	
	1	2.0-4.0	2.0-2.5	121.0	34.0	112.4	83.8	2.66
	2	2.0-4.0	2.5-3.0	² 90.2	39.3			
P-4.....	3	4.0-8.0	5.5-6.0	109.3	45.1	101.6	70.0	
	4	4.0-8.0	6.0-6.5	107.8	41.4	100.2	70.7	
	5	4.0-8.0	6.5-7.0	111.0	46.2	103.1	70.6	
	6	4.0-8.0	7.0-7.5	121.6	35.3	113.0	83.5	2.68
	7	4.0-8.0	7.5-8.0	127.0	39.7	118.0	84.5	

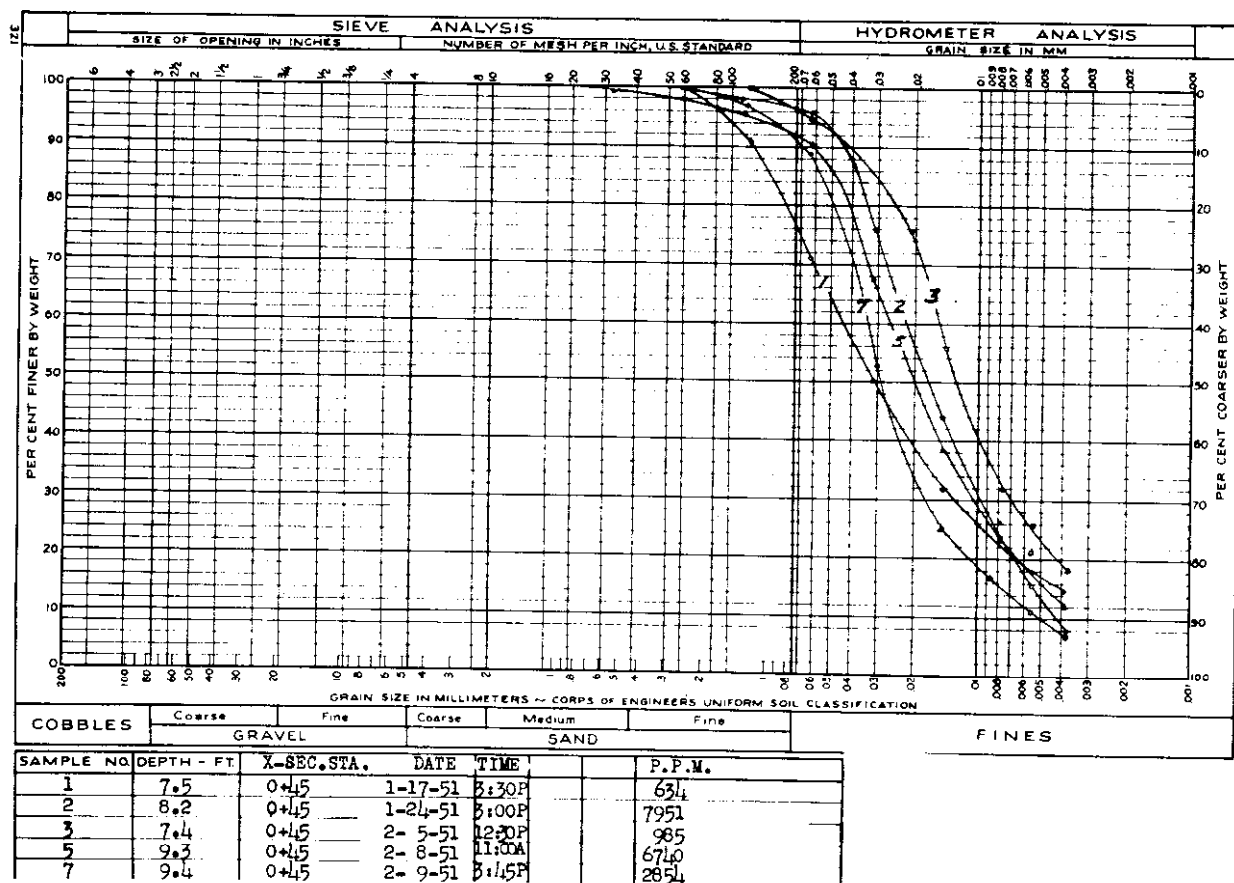
¹ Volume of sample cylinder was 67.16 cubic centimeters.² Denotes incomplete sample (no density possible).

FIGURE 7.— Gradation curves, suspended samples, Walla Walla River, Touchet, Wash.

condition typical of recent silt deposition. Appreciable further compaction can be expected.

The remaining storage space below normal pool upstream from range 1 is 3,390 acre-feet. Based on the indicated average annual rate of 860 acre-feet of sediment deposition, this area would fill in 3 more years or by the fall of 1965. However, the rate of deposition upstream from

range 1 will probably gradually decrease as the deposition approaches the main reservoir and a greater percentage of the sediment is carried out into the main reservoir. If possible variations of sediment load and stream runoff from year to year are considered, the remaining storage space essentially could fill in 1 year if a major flood occurred. In any case, the fore-

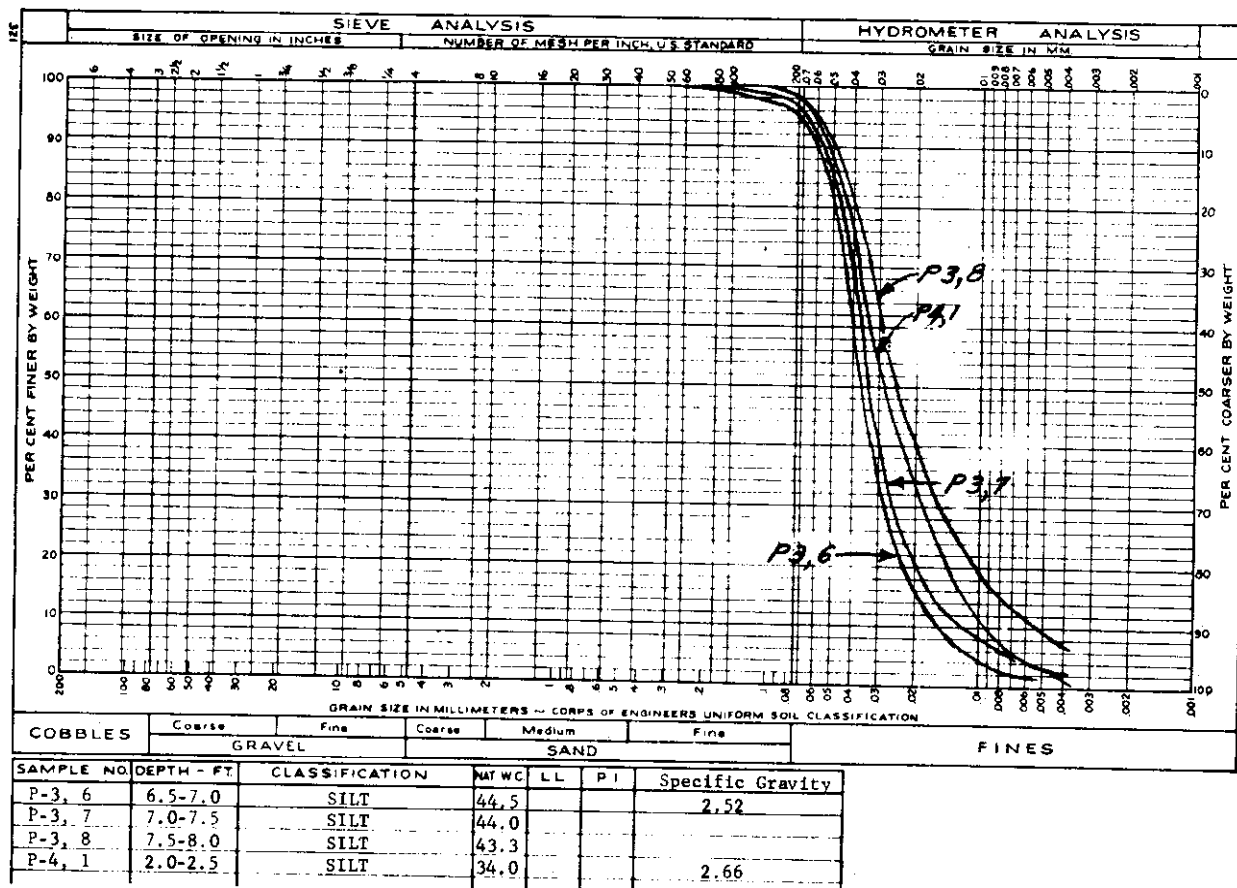


FIGURE 8. — Gradation curves, delta samples, Walla Walla River, Wallula, Wash.

going sediment measurements and analyses definitely show that the water storage area in the Walla Walla arm of McNary Reservoir will be essentially filled with sediment in a few more years, except for shallow pools and the channel area.

As yet, there apparently has been no appreciable deposition of sediment downstream from

range 1. Large amounts of deposition in this area would be of great concern to the Port District of Walla Walla (fig. 9), as it may affect navigable depths. Wave action is much more pronounced in the main reservoir, and this action is expected to distribute and keep the fine sediment transported by the Walla Walla River from rapidly depositing in the port area.

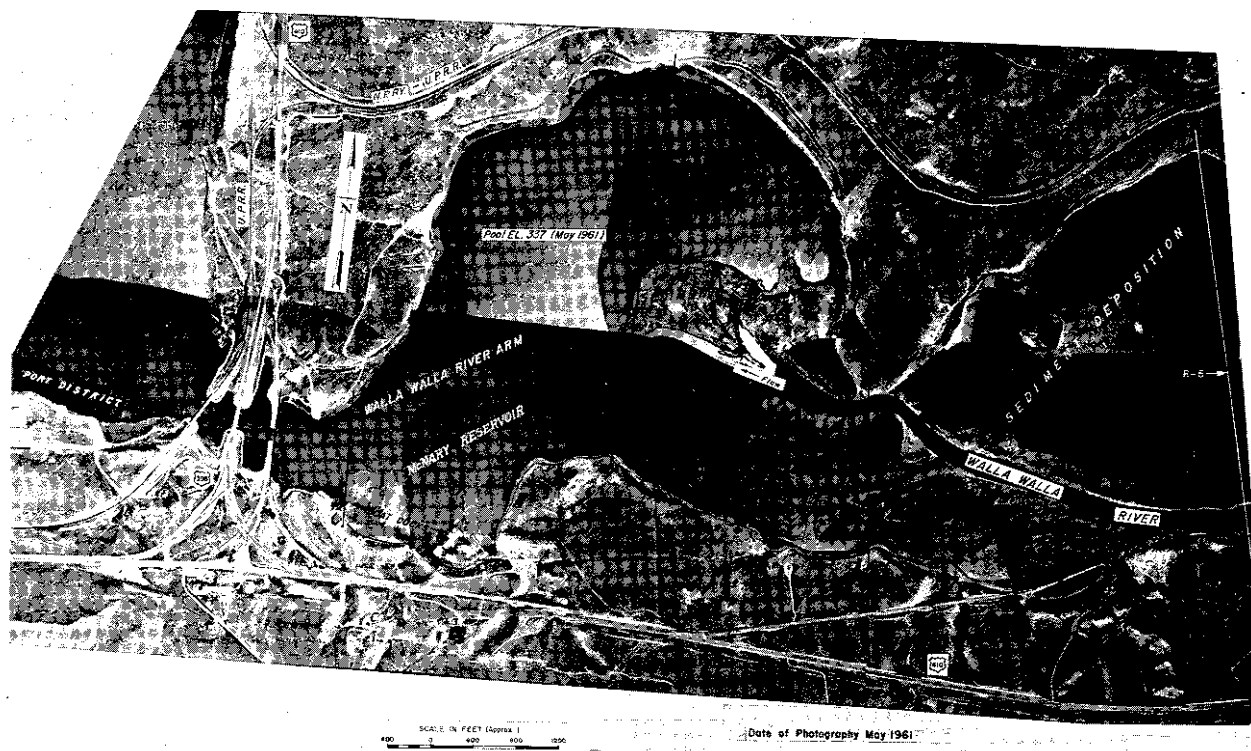


FIGURE 9. — Sediment range layout, Walla Walla River, ranges 1 to 5.

Reservoir Use Development, Walla Walla Arm, McNary Reservoir

Plans Considered

During the early planning for development and management of McNary Reservoir, the Walla Walla arm was selected as an outstanding recreational area. This area was considered

very desirable for development as a State park. A preliminary general development plan was prepared (fig. 17). Its chief assets were the apparent ample water area which was well protected from the prevailing southwest wind, access, good boat moorage sites, good beach areas, and scenic views. The principal features planned for the park include a public day use area, boating concession, organized group camp-



FIGURE 10. — Sediment range layout, Walla Walla River, ranges 5 to 9.

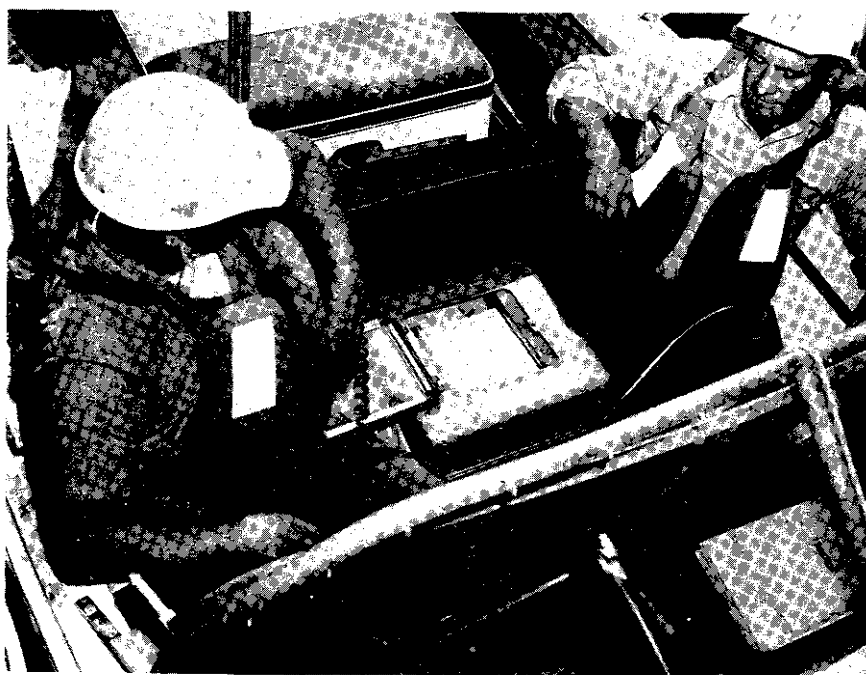


FIGURE 11. — Survey crew and equipment.



FIGURE 12.—Plane table being oriented over cut-in point and boat proceeding to range location to start sounding operations.

ing on the north and south shores, parking areas, launching ramp, and the usual beach and grounds development. Due to the sediment problem, however, recreational development thus far has been limited to a private boat headquarters, public launching ramps, picnic facilities, and wildlife areas. The development of a park has been abandoned by the State of Washington, but there continues to be strong local interest in maintaining use of as much of the area as practicable, particularly as a small boat harbor.

Investigations for Control of Sedimentation

Several plans for protection of the recreation areas and control of sedimentation on the Walla Walla River arm have been considered. The plans consisted of the construction of upstream sediment storage reservoirs on the Walla Walla and Touchet Rivers and the construction of parallel jetties to confine the river in a narrow channel designed to discharge the sediment into the main reservoir, with periodic maintenance dredging as required. Studies of the upstream sediment storage reservoirs proved to be infeasible. Other detailed studies showed the permanent effects of some of the proposed remedial measures could not be completely and accurately evaluated, and the costs were excessive when compared to probable benefits. Jetties for protection of the south shore development are undergoing further study and consideration, with a view to maintaining a small boat harbor.

Future Sediment Program

Important to the future planning and most feasible development of the Walla Walla River arm of McNary Reservoir is accurate and up-to-date information on sedimentation from the Walla Walla River. Because of the relatively short period of existing measurements, the rates of deposition and relationships to runoff are approximate estimates. Therefore, it is planned to continue periodic sediment surveys, including analyses of deposited material. Also, arrangements have been made for the U.S. Geological Survey to obtain regular suspended sediment samples near the mouth of the Walla Walla River.

Sediment deposition at the mouth of tributaries as they enter reservoirs is an important factor in planning reservoir use development at several other locations along the Columbia and Snake Rivers. Locations of such areas along the reservoirs that will be formed by John Day and Lower Monumental Dams are shown in figures 18 and 19, respectively. Information obtained from analyses of sediment measurements on the Walla Walla arm of McNary Reservoir has been useful in predicting amount and distribution of sediment at these locations. However, more information is needed and a program of obtaining regular suspended samples on the tributary streams has been instituted in cooperation with the U.S. Geological Survey.

Conclusions

The Walla Walla arm of the McNary Reservoir is rapidly filling with sediment. Based on five sediment surveys since 1953, the average annual rate of deposition is 860 acre-feet. The annual rate of deposition per square mile of Walla Walla River drainage area is 0.49 acre-foot. The estimated average annual sediment discharge of the Walla Walla River is 1,540 acre-feet.

The sediment deposition has greatly altered the original plans for recreational development of the area. Additional information and studies are required to determine the feasibility of providing protection and maintenance of a small boat harbor on the south shore, and the effects of port development.

A program of sediment surveys and suspended samples will continue in order to define more accurately rates and locations of sediment deposition in the Walla Walla arm and at other locations in reservoirs along the Columbia and Snake Rivers.

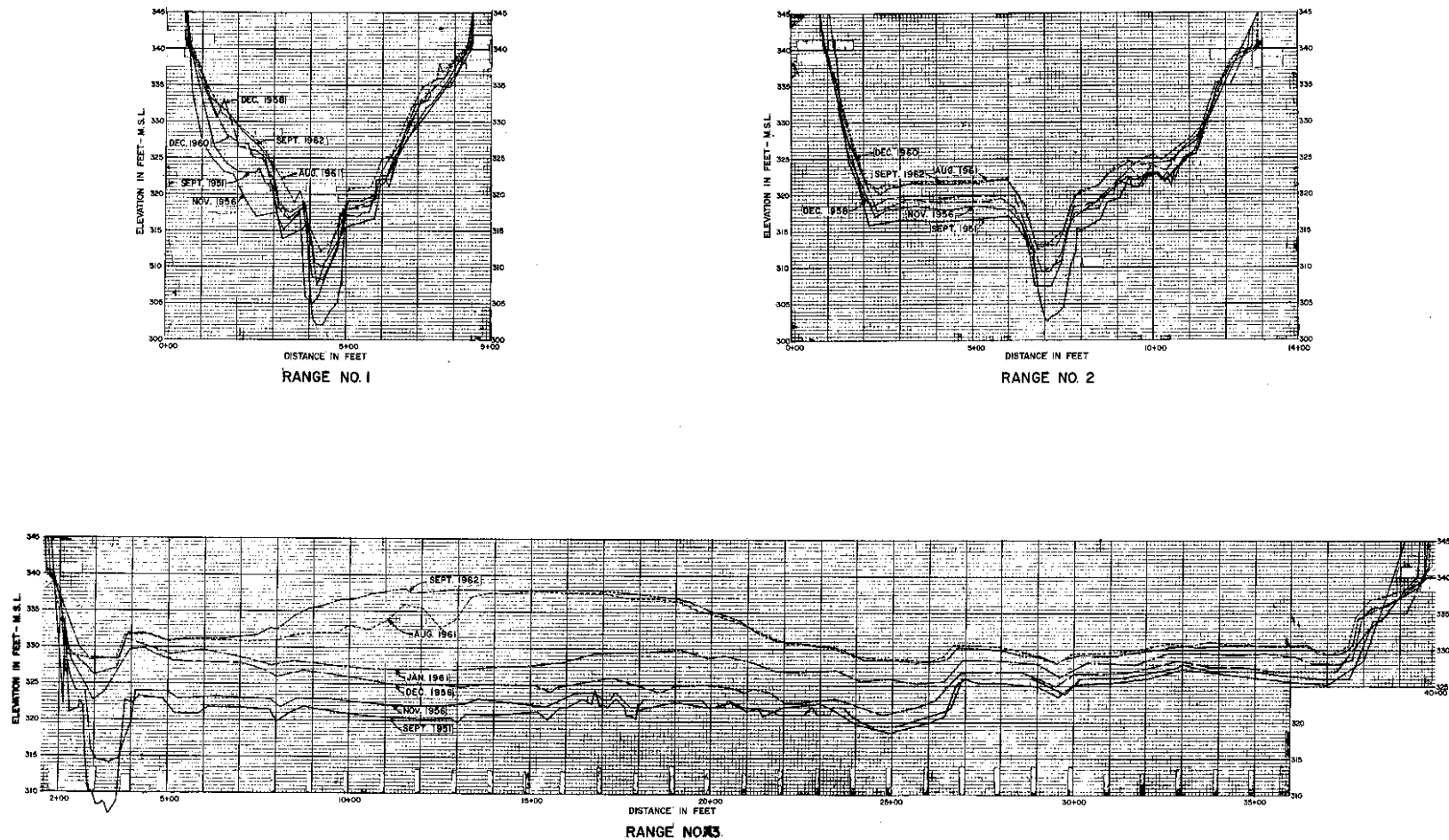
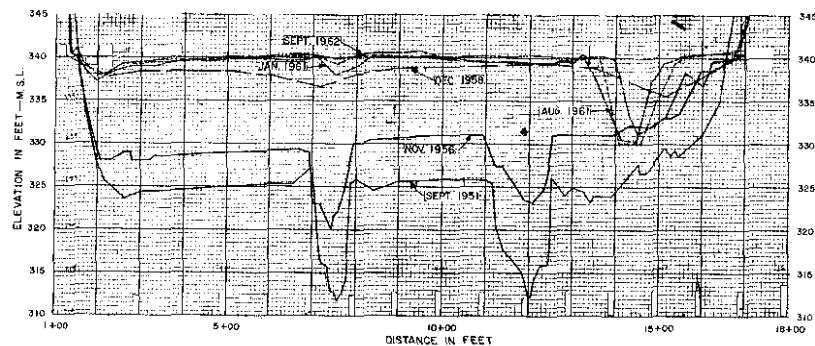
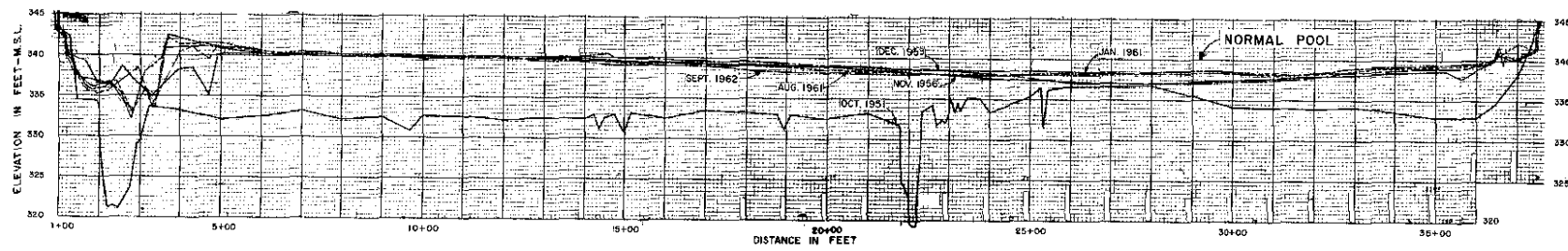


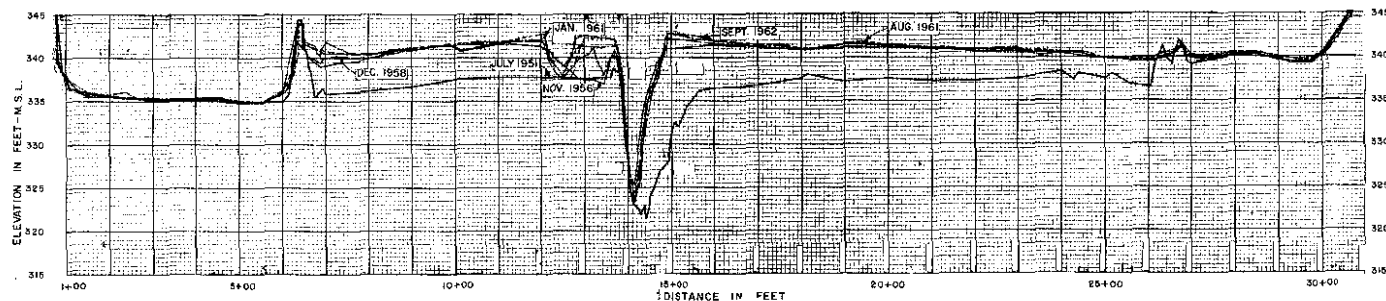
FIGURE 13.— Sediment range profiles 1 to 3, Walla Walla River.



RANGE NO. 4



RANGE NO. 5



RANGE NO. 6

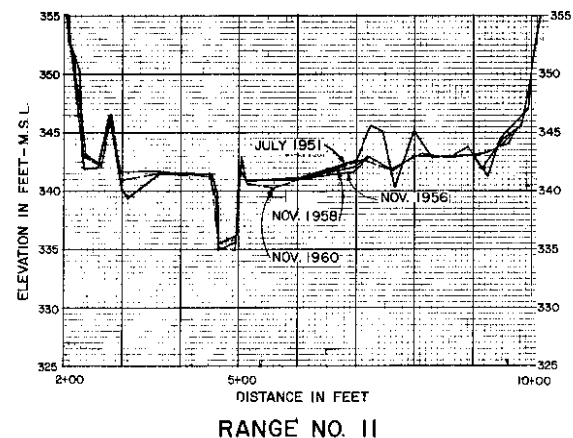
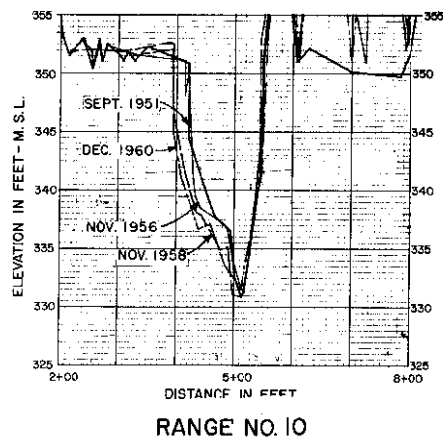
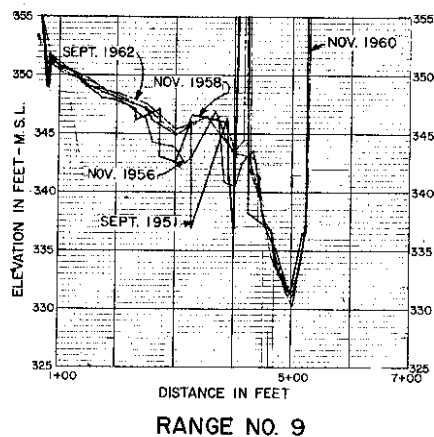
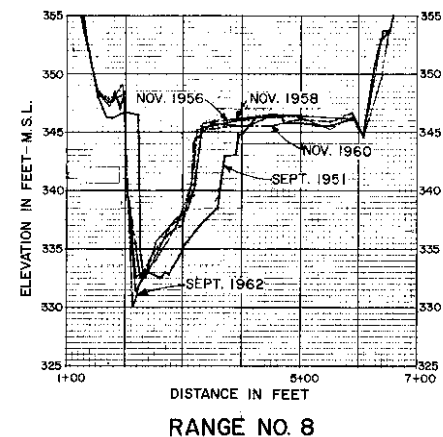
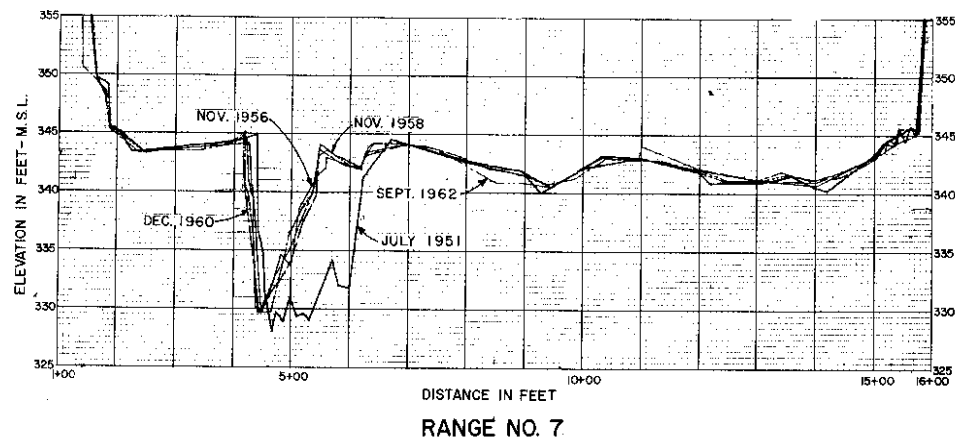


FIGURE 15. — Sediment range profiles 7 to 11, Walla Walla River.

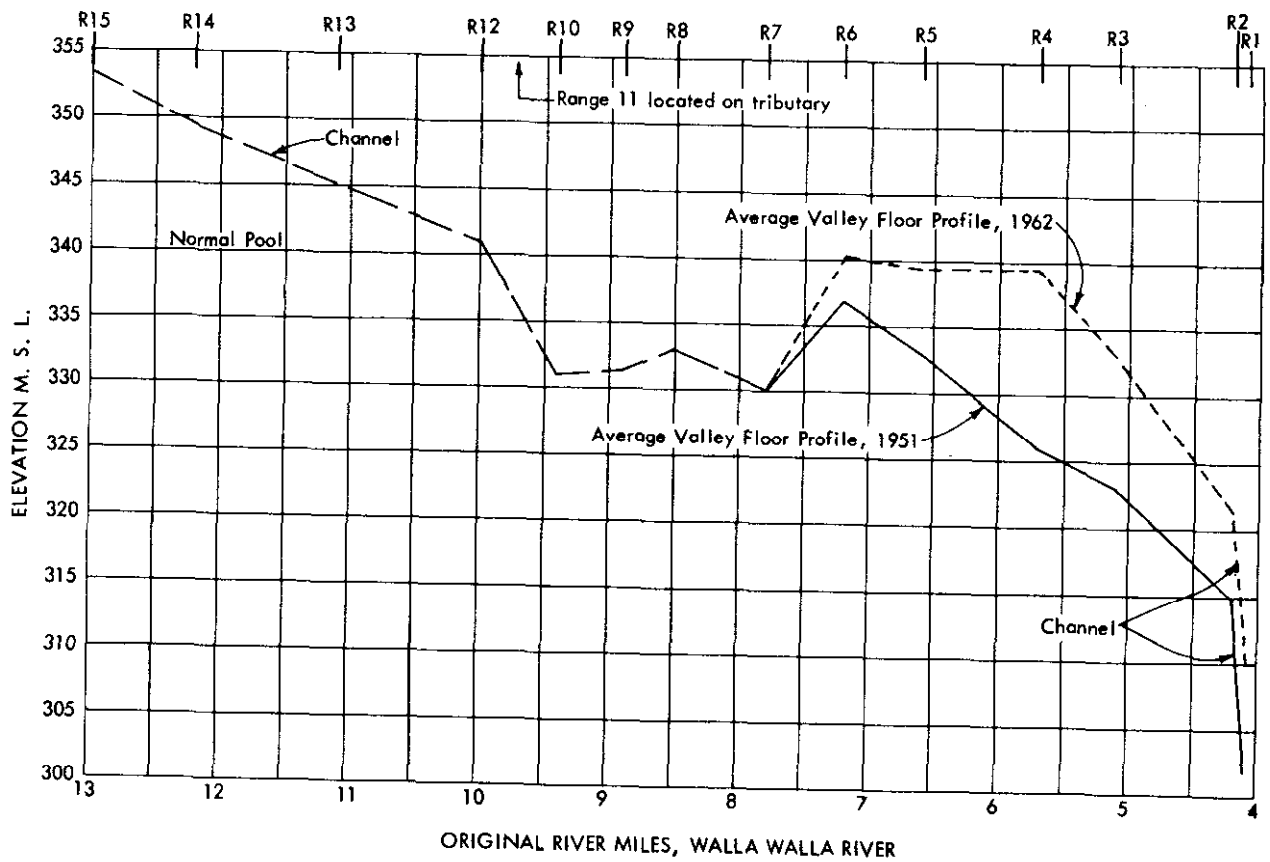


FIGURE 16. — Longitudinal profile sediment ranges, Walla Walla River.

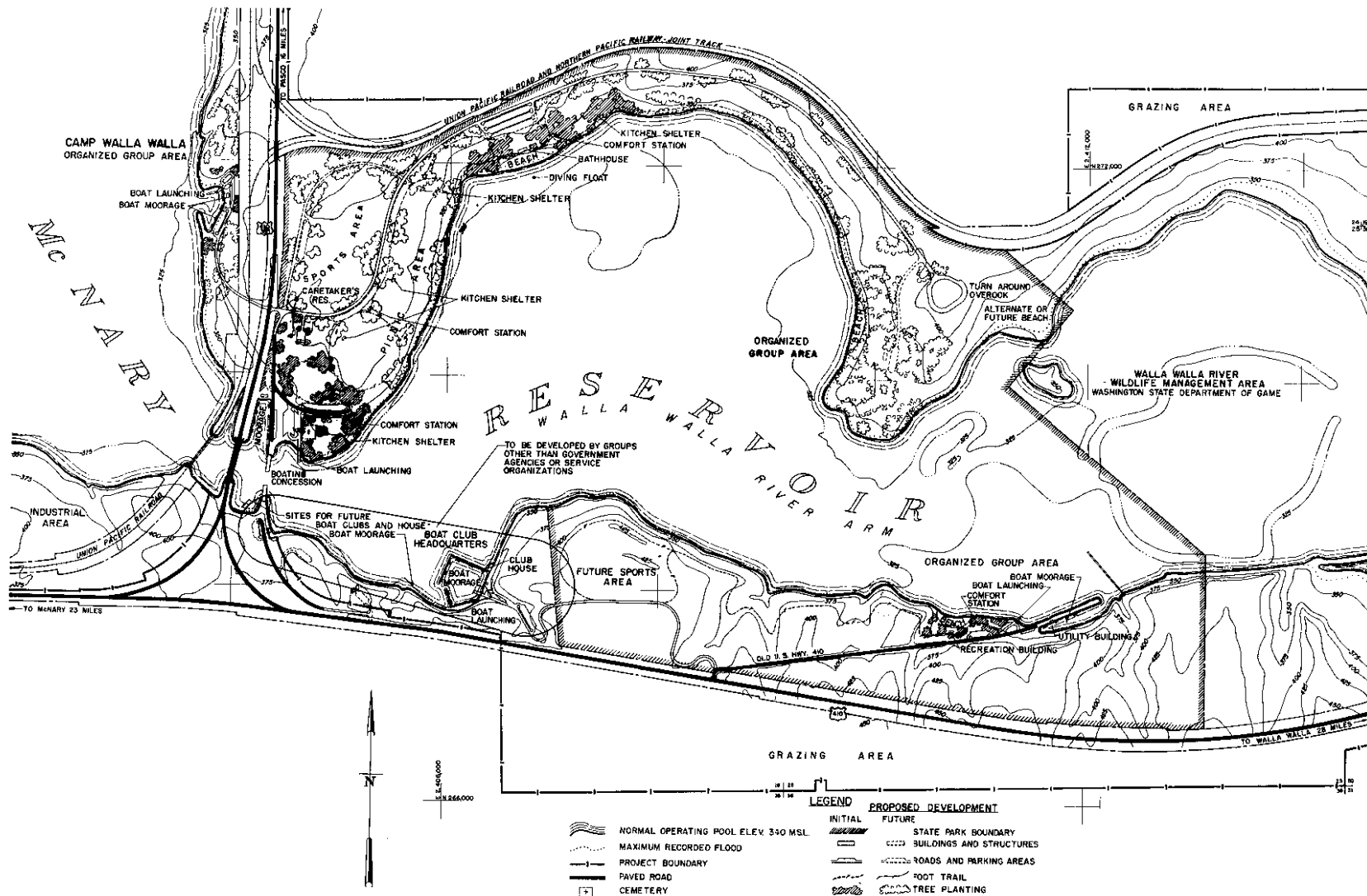


FIGURE 17.— Preliminary development plan for State park.

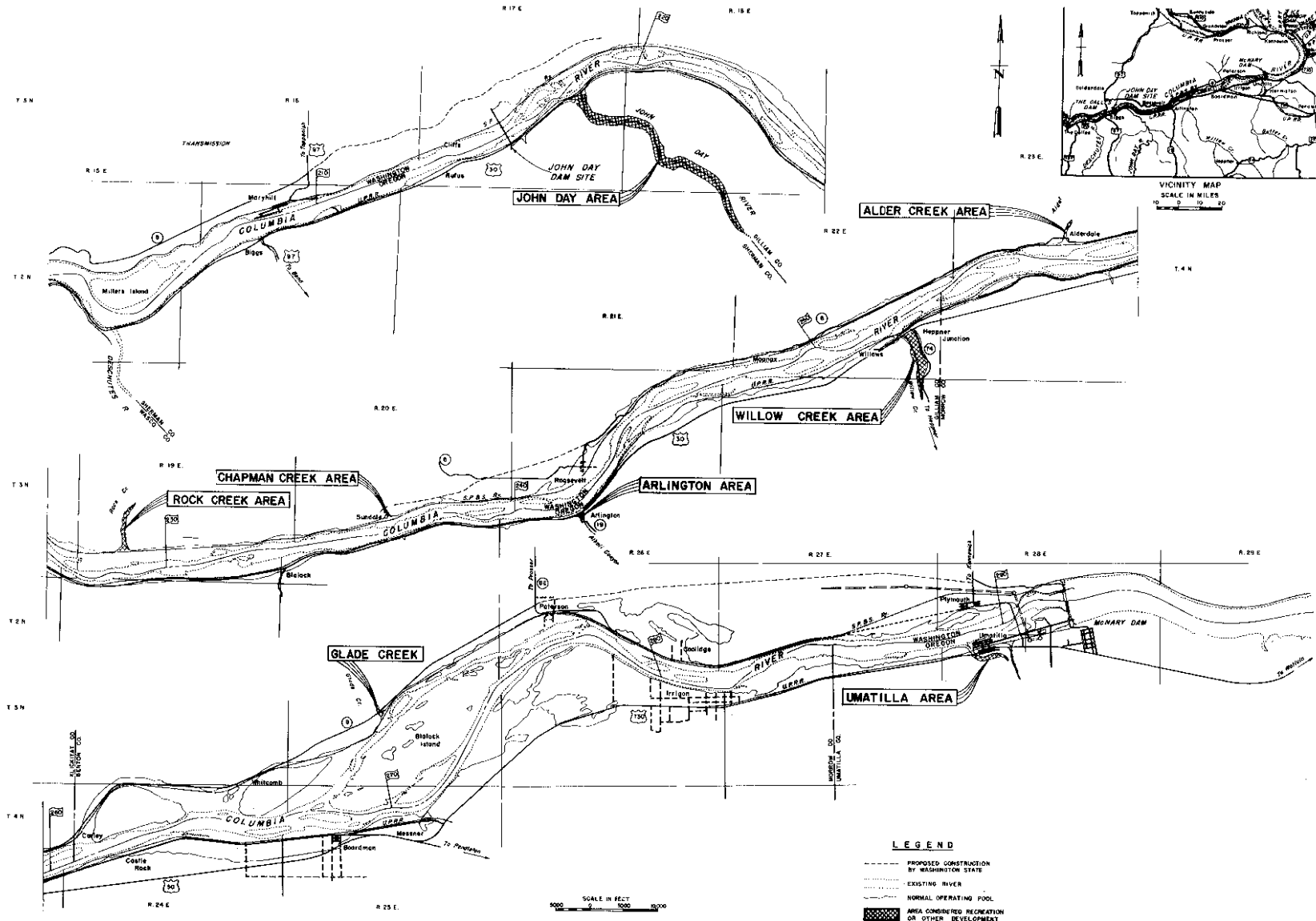
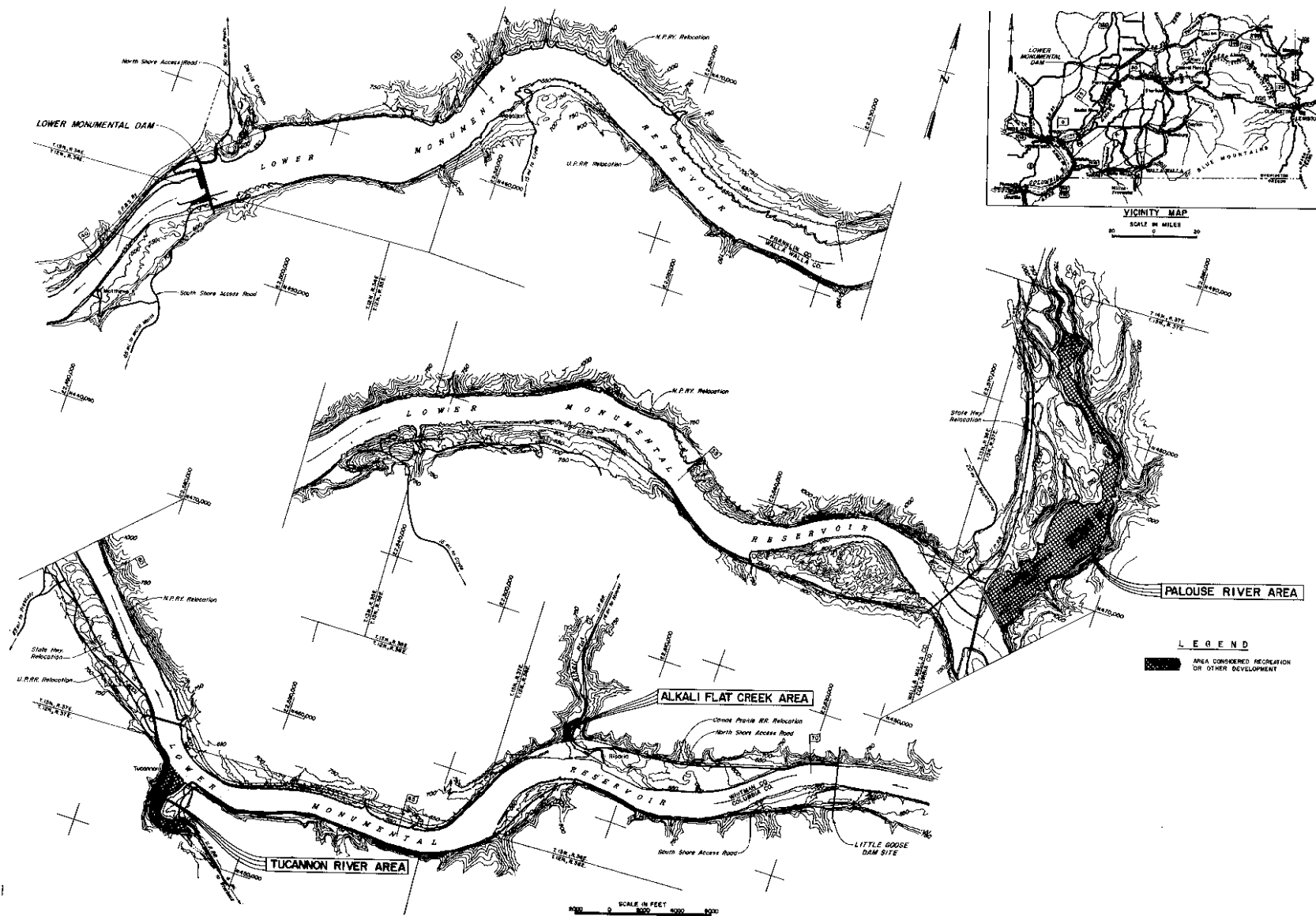


FIGURE 18. — John Day Reservoir map.



CHARACTERISTICS OF SEDIMENT DEPOSITS ABOVE CHANNEL STRUCTURES IN POLACCA WASH, ARIZ.

[Paper No. 80]

By R. F. HADLEY, *geologist, U.S. Geological Survey, Denver*

Abstract

Deposition caused by the construction of dams on Polacca and Wepo Washes since 1945 is estimated at 7,500 acre-feet. Longitudinal profiles were surveyed above each dam on Polacca Wash. The present channel gradients on the sediment deposits varies from 0.0005 ft./ft. to 0.0037 ft./ft. The gradients of these channels prior to dam construction ranged from 0.0040 ft./ft. to 0.0058 ft./ft.

Of the total sediment deposited behind the structure approximately 8 percent is located above the spillway level and aggradation was observed some distance above the anticipated influence of the dams.

Introduction

Many of the alluvial valleys in southwestern United States have been trenched by deep arroyos in the past 70 years. Many others have just begun to degrade; a few have retained a continuous, relatively flat valley floor or are

aggrading. These various stages of erosion and alluviation often can be observed in contiguous drainage basins. Conservation measures, such as construction of dams and water-spreading dikes, have been initiated in some trenched valleys in an effort to restore the usefulness of the valley floor for grazing and floodwater farming by promoting aggradation of the arroyo.

The primary objective of this report is to describe the extent of channel and flood plain aggradation induced by dams and the effect of these deposits on the longitudinal profile of the arroyo. The study was done in the Polacca Wash drainage basin in northeastern Arizona (fig. 1).

Polacca Wash and its principal tributaries, Wepo Wash and Kean's Canyon Wash, are typical of the many streams that drain Black Mesa in the Navajo and Hopi Indian Reservations. In the upper third of the basin the streams are narrow and are confined in deeply incised canyons. Near the village of Polacca, they become wider, and flow in broad, relatively flat valleys.

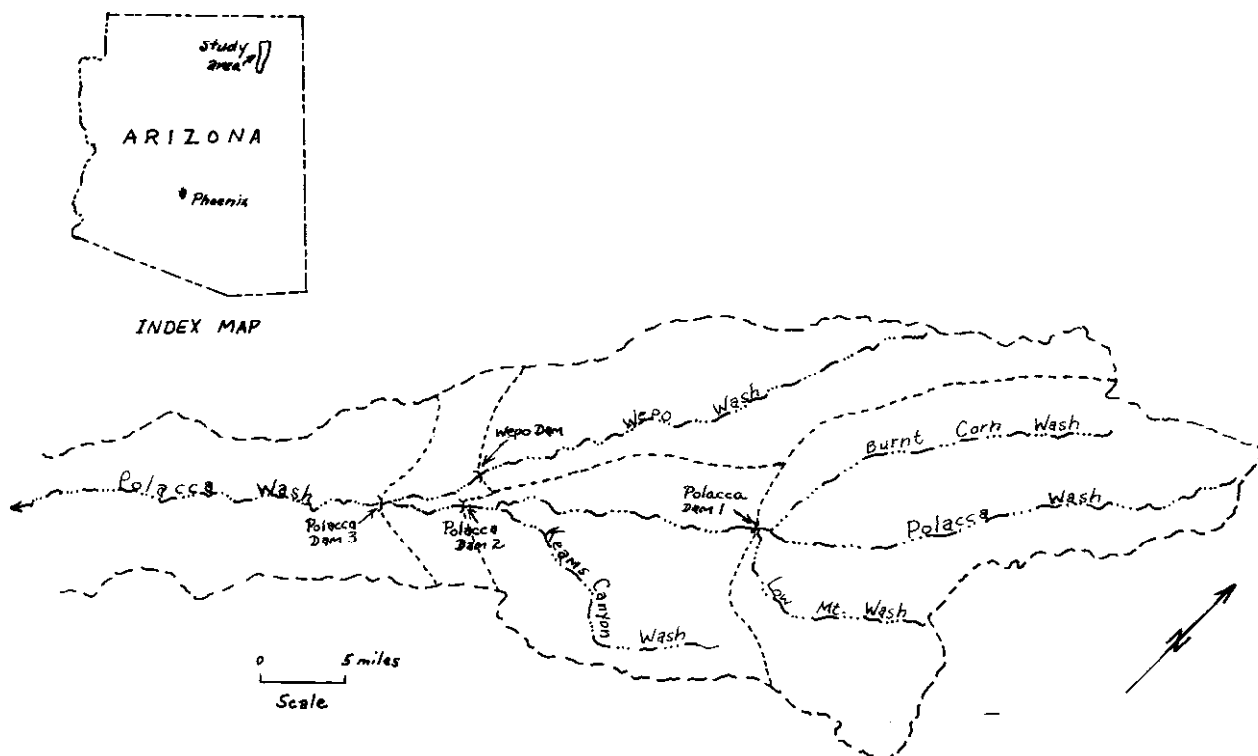


FIGURE 1. — Polacca Wash basin, showing location of dams.

Windblown sand on the valley floors obscure many of the smaller tributaries. In the central and lower reaches of the drainage basin the tributaries are generally aggraded, forming alluvial fans that obliterate the confluence with main channels. For most of its length the present Polacca Wash channel is deeply entrenched in the valley alluvium.

Deposition Caused by Channel Structures

The amount of sediment deposited in the channels and on the valley floors of Polacca and Wepo Washes caused by construction of the diversion dams and waterspreaders was calculated from surveys of the longitudinal profile and valley cross sections and mapping on aerial photographs. Limitations must be placed on the precision of these calculations because of the lack of complete data on the original profile of the streambed and shape of the channel. However, measurements of the channel and valley downstream from the dam probably provide an accurate representation of the original reservoir contours above the dam. Also, by extending the longitudinal profile from some point downstream from the dam to a point where the extension intersects the channel floor above the reservoir deposits, the original profile is closely approximated. The channel profile and valley shape do not vary greatly in a distance equal to that affected by a dam. Using the reconstructions to provide data on valley shape and slope, the volume of sediment was calculated.

Between about 1900 and 1945 the channel of Polacca Wash eroded to a depth of about 40 feet in the reach near the village of Polacca. In the years 1945 to 1953 three dams were constructed on Polacca Wash in this reach and another near the mouth of Wepo Wash. (See fig. 1.) The purpose of these structures is to divert flood flows onto the valley floor for irrigation and to promote deposition of sediment in the arroyo.

Polacca Dam 2 was constructed in 1945 and is located about 4 miles upstream from the mouth of Wepo Wash. The crest of the dam is 47 feet above the bed of the arroyo, which is about 40 feet deep in this reach. Sediment has almost completely filled the channel behind the dam for a distance of 4,000 feet. About 3,000 acre-feet of sediment have been deposited in the reservoir since 1945 at an annual rate of about 0.5 acre-foot per square mile. Since 1953 storage in Polacca Dam 1 has reduced the sedimentation of the reservoir of Polacca Dam 2.

About one-quarter mile below the mouth of Wepo Wash is Polacca Dam 3, constructed in 1946. At this site Polacca Wash is entrenched 38 feet into the valley floor. The crest of the dam is 48 feet above the bottom of the wash and subsequent deposition of sediment behind this barrier completely filled Polacca Wash for 9,030 feet upstream. The maximum depth to which sediment has accumulated at the dam is 38 feet. The average thickness of sediment behind the dam is 21 feet (table 1), and the total volume of sediment deposited back of Polacca Dam 3 is 1,090 acre-feet. The average annual rate of

TABLE 1.—*Estimates of deposition caused by construction of dams on Polacca and Wepo Washes, Ariz.*

Name of structure	Surface area of reservoir	Maximum thickness of deposition	Average thickness of deposition	Volume of sediment deposition	Area of water-spreading	Volume of sediment deposition on waterspreading area ¹	Drainage area
	<i>Acres</i>	<i>Feet</i>	<i>Feet</i>	<i>Acre-feet</i>	<i>Acres</i>	<i>Acre-feet</i>	<i>Square miles</i>
Polacca Dam 1.....	180	15	11	1,900	260	80	338
Polacca Dam 2.....	133	38	23	3,000	586	290	511
Polacca Dam 3.....	53	38	21	1,090	1,710	210	757
Main Wepo Dam.....	26	15	7	190	920	690	197

¹ Estimates for deposition on waterspreading area are probably less than actual amounts because much of the sediment has been disturbed in repair of training dikes.

sediment accumulation is 0.8 acre-feet per square mile for the period 1946-57. Polacca Dam 3 has served primarily to divert runoff to the waterspreading area along the south bank of Polacca Wash below the damsite, and to prevent further deepening of Polacca Wash. This dam caught all the overflow from Polacca Dam 2 before construction of a training dike next to the arroyo bank and any uncontrolled flow from Wepo Wash. The overbank deposition has raised the level of the valley floor 3 feet in the reservoir area and channel deposits extend to an elevation 15 feet above the spillway.

About 14 miles upstream from Polacca a third dam was constructed on the main channel and has been designated Polacca Dam 1. This dam was completed in 1953 and is located just below the mouth of Polacca Wash with Burnt Corn Wash and Low Mountain Draw (fig. 1). The arroyo has been completely filled above the structure for a distance of 2,300 feet, and channel deposits extend upstream for 7,500 feet to the mouth of Burnt Corn Wash.

The channel downstream from Polacca Dam 1 is a rectangular-shaped arroyo and most of the storage in the reservoir was confined to a simi-

lar channel upstream. The maximum thickness of deposition in the reservoir is estimated to be 15 feet and the average is about 11 feet. The volume of sediment deposition in the reservoir is calculated to be 1,900 acre-feet, and the annual rate of deposition to be 1.4 acre-feet per square mile. The drainage area above Polacca Dam 1 is about 340 square miles and, although it is not certain how much area is controlled by structures, an inspection of part of the basin, which is accessible by roads, revealed no appreciable diversion or storage.

On Wepo Wash the main dam was constructed in 1944 about $1\frac{1}{4}$ miles above the mouth. The crest of the dam is 25 feet above the floor of the Wash and has caused channel deposition for a distance of 5,360 feet upstream. At the dam, sediment has accumulated to a maximum depth of 15 feet. The total volume of sediment accumulated above the main Wepo Dam is 190 acre-feet, with an average thickness of 7 feet (table 1). Downstream from the main dam are two training dikes that divert overflow from the dam away from the main channel of Wepo Wash. Much of the sediment retained in the waterspreading areas below the main Wepo Dam has been utilized to construct additional training dikes, thus making any determination of the total amount of sediment deposited uncertain. The average annual rate of sediment accumulation above the main Wepo Dam is 0.1 acre-foot per square mile for the period 1944-57.

Some comparisons may be made between the suspended-sediment load carried by the Little Colorado River, as measured near Grand Falls, Ariz., and the estimates of sediment trapped by the structures on Polacca Wash. The Little Colorado River basin has a drainage area of 21,200 square miles above Grand Falls and available records of suspended-sediment load were complete there for 9 years at the time of the study (1933, 1948-55). The suspended-sediment loads range from 1,000 to 13,900 acre-feet annually, with an average for the 9-year period of 5,640 acre-feet. Distributed over the drainage basin, this figure represents an average annual sediment yield of about 0.30 acre-foot per square mile.

Since 1945, when the first dam was constructed, the structures in Polacca Wash basin, including waterspreading systems, have trapped approximately 7,500 acre-feet of sediment. The drainage area above these structures comprises approximately 950 square miles. The annual sediment yield per unit of drainage basin has been discussed previously and, because the struc-

tures were completed at different times, a mean rate of sediment yield, 0.7 acre-foot per square mile, will be used for comparison with the little Colorado River.

The data seem to indicate that the Polacca Wash area has a higher unit rate of sediment yield and consequently more severe erosion than other areas within the Little Colorado River basin, with a mean annual rate of sediment yield nearly three times that of the river at Grand Falls. It has been shown in other studies,¹ however, that in similar drainage basins the sediment yield decreases with increase in size of drainage area. Therefore, areas of similar size and geology in the Little Colorado River basin may have sediment yields comparable to Polacca Wash.

Relation of Channel Slope to Deposition

Gradients were measured on the deposits upstream from the dams to the apparent limit of deposition. Results of these profile surveys and comparisons with approximations of the original channel profiles are shown in table 2. The

TABLE 2.—Comparison of original and present channel gradients above Polacca Wash diversion dams

Structure	Original gradient ¹ Ft./ft.	Present gradient Ft./ft.
Polacca Dam 1.....	0.0040	0.0005-0.0037
Polacca Dam 2.....	.0041	.0016
Polacca Dam 3.....	.0058	.0010

¹ Original gradients as determined from survey of the channel and reconstruction of profile.

channel above Polacca Dam 1 has been filled to spillway level for a distance of 1,000 feet above the dam and has a gradient of 0.0005 ft./ft. and a second wave of sediment with a steep front $3\frac{1}{2}$ feet high is encroaching on the dam. Therefore, two values for the gradient are shown in table 2. The arroyo in Polacca Valley has been aggraded to an elevation higher than spillway above the three dams that have been constructed and overbank deposition has raised the valley floor in some places. The appearance of the channel and flood plain upstream from each of these dams is quite similar to valleys in the Polacca Wash area that are aggrading naturally.

In other studies of the slope of reservoir deposits similar results were found. In the Mexican Springs experimental area near Gallup, N. Mex., several dams were constructed in 1936 to study the effects of waterspreading. Topography of the arroyo channel and valley floor were mapped in detail at that time by the Soil Conservation Service. The gradient of the arroyo floor ranged from 0.0071 ft./ft. to 0.0105 ft./ft. and averaged 0.0075 ft./ft. In 1949 when

¹ HADLEY, R. F., and SCHUMM, S. A. SEDIMENT SOURCES AND DRAINAGE BASIN CHARACTERISTICS IN UPPER CHEYENNE RIVER BASIN. U.S. Geol. Survey Water-Supply Paper 1531-B: 137-198. 1961.

the arroyos had been completely aggraded behind the dams, the gradients of the alluvial fills were measured by the Geological Survey.² The gradients ranged from 0.0025 ft./ft. to 0.0044 ft./ft. and averaged 0.0037 ft./ft.

The Bureau of Reclamation conducted a study in 1952 and 1953 of the amount and location of sediment behind Zion Dam on the Little Colorado River near St. Johns, Ariz.³ The gradient of the original channel from the dam upstream to the elevation of spillway crest, as determined by borings, ranged from 0.00070 ft./ft. to 0.00148 ft./ft. and averaged 0.00123 ft./ft. The gradient of the channel on the alluvial fill ranges from 0.0053 ft./ft. to 0.00070 ft./ft. and averages 0.00058 ft./ft.

The gradient of the original channel in each of the areas described was much steeper than the channel that now flows at or near the level of the valley floor. The data also indicate that the slope is related, in some degree, to the size of drainage area above the point of measurement. In figure 2 the relation between stream

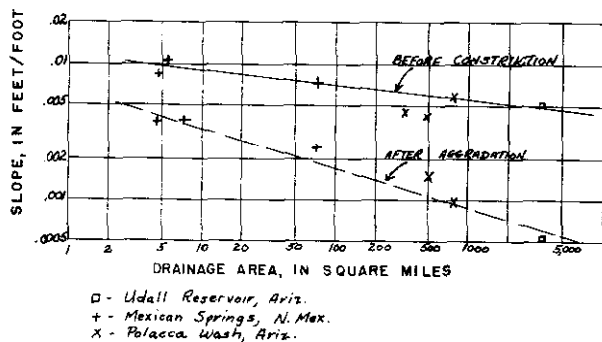


FIGURE 2.—Relation of drainage area to channel slope for several areas in Arizona and New Mexico.

gradient and drainage area is shown for the areas in Polacca Wash, Mexican Springs, and Little Colorado River. The longitudinal profiles of these channels prior to construction of the dams probably were representative of ephemeral channels in equilibrium or quasi-equilibrium. A similar relation for uncontrolled ephemeral channels is shown for New Mexico.⁴

² PETERSON, H. V., and SNYDER, C. T. SEDIMENT DEPOSITION IN THE MEXICAN SPRINGS EXPERIMENT STATION, NAVAJO INDIAN RESERVATION, N. MEX. U.S. Geol. Survey Memo. Rpt. 1949.

³ U.S. BUREAU OF RECLAMATION. REPORT ON SEDIMENT DEPOSITION ABOVE ZION DAM, LITTLE COLORADO RIVER BASIN, ARIZ. 1955. [Processed.]

⁴ LEOPOLD, L. B., and MILLER, J. P. EPHEMERAL STREAMS—HYDRAULIC FACTORS AND THEIR RELATION TO THE DRAINAGE NET. U.S. Geol. Survey Prof. Paper 282-A: 27. 1957.

The gradients of the channels developed on the alluvial fills are also shown in figure 2. The gradient at each station is less than the original gradient and decreases with an increase in size of drainage area. The longitudinal profile is dependent primarily on median size of alluvial material, density of vegetational cover, or any number of hydraulic factors.

The relation between channel slope and median diameter of bed material in the aggraded channels is presented in figure 3. There is some

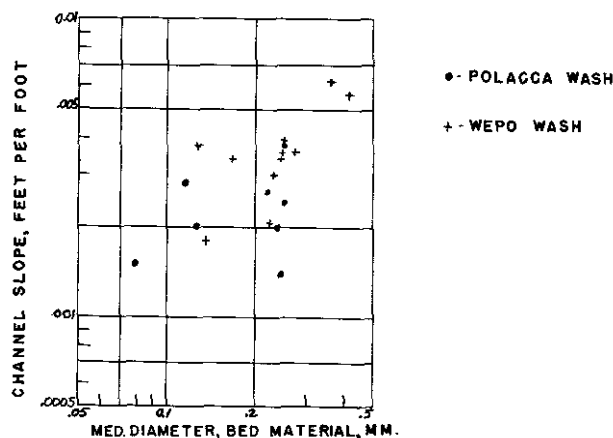


FIGURE 3.—Relation of median diameter of bed material to channel slope.

indication that an increase in size of bed material is accompanied by an increase in channel slope. The narrow range of median diameters, however, prevents defining the relationship more precisely.

Location of Reservoir Deposits

The location of sediment deposits in the reservoir area and channel above a dam are of prime importance to engineers responsible for design and allocation of space for dead storage of sediment. The effect of an artificial base level produced by the construction of a dam will result in marked changes in the stream channel and flood plain for some distance upstream above the elevation of the spillway, which is the new control.

In the studies of the Polacca Wash dams estimates were made of the volume of sediment trapped and of the original profile of the channel reach within the influence of the dam. From these data we may arrive at some approximate figures for the quantity of sediment that has been deposited above the elevation of the spillway control and to what elevation above the spillway deposition occurs.

Estimates of deposition above the present

spillway elevation were made from reservoir cross sections and aerial photographs. The results of these computations for the Polacca Wash structures are shown in table 3.

TABLE 3.—*Location of reservoir sediment above Polacca Wash diversion dams*

Dam	Total sediment trapped	Sediment deposited above spillway level	Percentage of total	Height of deposition above spillway
	<i>Acres-feet</i>	<i>Acres-feet</i>		<i>Feet</i>
Polacca Dam 1.	1,900	100	5	12.5
Polacca Dam 2.	3,000	275	9	13
Polacca Dam 3.	1,090	108	10	15

The data in table 3 show that of the total sediment trapped in the three reservoirs, 5 to 10 percent lies above present spillway elevation

and also that deposition occurs from 12.5 to 15 feet above the spillway elevation. In the study made by the U.S. Bureau of Reclamation on Zion Dam near St. Johns, Ariz., it was concluded that about 10 percent of the reservoir sediment lies above the present spillway crest elevation.

In summary, we may conclude that a change in stream regimen affected by a rising base level such as a dam will reduce the channel slope and cause aggradation upstream to a higher elevation than that of the channel control. The extent of this deposition may be affected by valley width, channel slope, particle size of available material, and influence of riparian vegetation. The relative importance of each of these factors has yet to be determined.

RESERVOIR SEDIMENTATION SURVEY METHODS IN THE U.S. ARMY ENGINEER DISTRICT, VICKSBURG, MISS.

[Paper No. 81]

By OMEGA H. SHAMBLIN, *engineer, U.S. Army Engineer District, Vicksburg, Miss.*

Synopsis

The Vicksburg District has developed a practicable method¹ for performing sedimentation surveys which produces a degree of accuracy that is consistent with the conditions existing in that district, which has experienced only relatively minor siltation. The method is also used in the periodic resurveys of these ranges.

In this development, the major departure from conventional survey methods is in the positioning of soundings along the ranges. Briefly, a taut steel piano wire is anchored along the exact alinement of each range. As the sounding boat proceeds along the range, the distances are automatically determined at 20-ft. intervals by means of a calibrated measuring wheel that is encircled by the wire, the lake bottom elevations are simultaneously recorded by a supersonic depth recorder, and alinement is maintained by radioed instructions from a transitman on the bank to the sounding boat operator.

Alinement and distance errors are considered to be within 2 ft. Plotted results of ranges profiled by this method have been compared with direct leveling profiles surveyed before the reservoir was filled. These comparisons indicate an accuracy in elevation of ± 0.5 ft. in depths up to 23 ft., and ± 1.0 ft. in depth up to 180 ft.

¹ This survey method was published in the January 1961 issue of *International Hydrographic Review*. This article is a revision of that publication, to reflect the changes in methods and equipment since the original article was prepared.

General

To determine the rate of silting in reservoirs, the U.S. Army Engineer District, Vicksburg, uses a system of permanently marked ranges that are usually established either during or immediately after construction of the dam. These ranges are surveyed periodically to obtain comparative profiles, which indicate the rate of silting. Figure 1 is a range layout for one of the reservoirs.

Previous Survey Methods

Prior to 1950 the permanent ranges were established and profiled by conventional direct-leveling survey methods before the reservoirs were filled. The original range layouts were to third-order accuracy. (Horizontal position closure $1/5,000$, vertical closure 0.05 ft. $\sqrt{\text{length of loop in miles}}$.)

Resurveys of these ranges were made by direct leveling and taped distances on the portions of the ranges above the water surface, and by lead-line soundings below the water surface. Horizontal positions for the soundings were determined by visual signals from the sounding boat, as it traveled along each range line, and by turning angles to the boat at each sounding with a transit at a convenient location on the shore.

This method was used for a resurvey of Sardis Reservoir in northern Mississippi in 1943. The results of the underwater surveys were not considered satisfactory, because of the errors in the horizontal position of each sounding

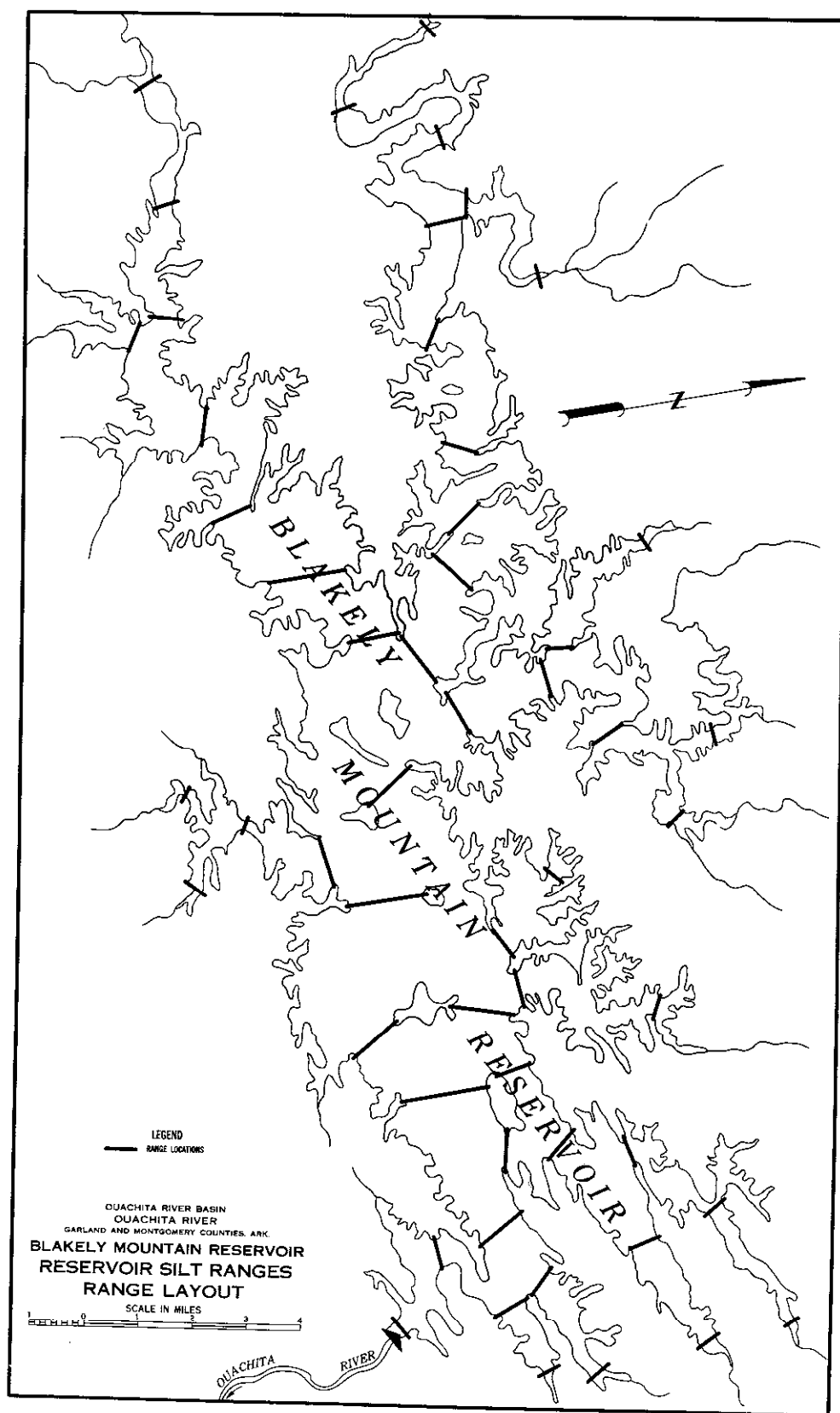


FIGURE 1.—Layout of reservoir deposition ranges of mountain reservoir.

and the inability to determine the top of the silt deposit with the sounding weight. These inaccurate results caused the Vicksburg District to search for a more reliable survey method.

Improved Method for Resurveys

During the period 1946-47, a new procedure for the underwater survey was developed and pilot models of specialized equipment were made. This improved survey method was first used during a resurvey of Arkabutla Reservoir, also in northern Mississippi, in 1947-48. The results of this survey proved that the new method would produce results that were considerably more accurate than those of the methods previously used.

In brief, the improved method consists of stretching a wire on range across the reservoir. A calibrated distance-measuring wheel traveling along this wire measures distances simultaneously as the depths are obtained with a supersonic depth recorder. Alinement directions are transmitted to the sounding-boat operator by radio from the transitman located at a

point on the range as the boat proceeds along the range. Figure 2 shows the sounding boat and equipment.

Improved Method of Surveys for Establishing and Profiling Ranges

Before the construction of Blakely Mountain Dam in west-central Arkansas, it was evident that conventional surveys for establishing and profiling the permanent-range system in the rugged and mountainous terrain in which the reservoir was located would be unduly expensive. It was therefore decided to:

- (1) Delay establishing the permanent ranges until after water was impounded in the reservoir;
- (2) Eliminate expensive traverse in the mountains by locating the permanent-range markers by ties to aerial photographs;
- (3) Use the water surface of the lake as a datum plane to eliminate the need for direct leveling for vertical control; and

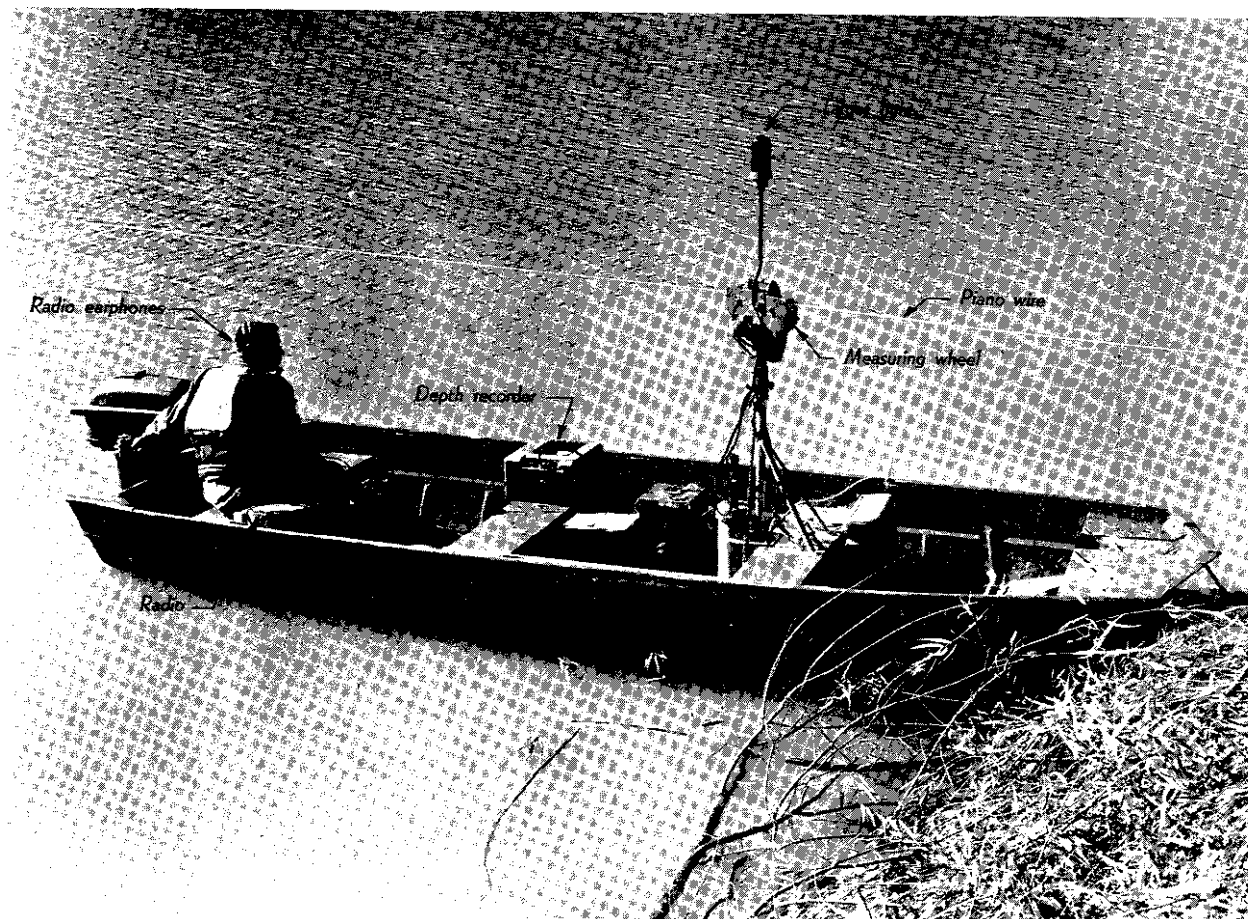


FIGURE 2. — Sounding boat and equipment used in sedimentation surveys.

- (4) Use the improved method developed for resurveys in making the initial underwater survey of the permanent ranges.

A summary description of the survey method follows.

Horizontal Control

Horizontal positions of the permanent-range markers are obtained by ties to aerial photographs. This eliminates traverse tie-lines between ranges and ties to existing control.

Vertical Control

The water surface in the reservoir is used as a datum plane for establishing elevations on the permanent-range markers. In determining the datum plane for the water surface in the Blakely Mountain Reservoir Survey, several gages were located at strategic points around the reservoir. These gages were tied to existing first- and second-order benchmarks and were read twice daily during the period of the survey. It was found that water-surface fluctuations were constant at all gages and that the differences in water-surface elevations at the various gages were never more than two-tenths of a foot. This eliminated the necessity for level tie-lines between ranges and ties to existing control.

Range Profiles

The portions of all ranges above the reservoir level are traversed and profiled by conventional direct-leveling survey methods. The underwater portion of each range is profiled with the special equipment and improved method described below.

Special Equipment

Special equipment needed for this operation includes supersonic depth recorder, distance wire and reel, measuring wheel, radios, and wire splicer.

Supersonic depth recorder

The depth recorder used on the Blakely Mountain Reservoir Survey was manufactured commercially.

The specifications on the outboard oscillator were:

Sound frequency.....	14 kc./sec.
Beam width.....	not exceeding 30°
Sounding rate.....	200 per minute
Theoretical accuracy.....	3 in. in 150 ft.

The depth recorder used on later surveys is a light, more easily portable instrument (see fig. 2). The specifications on the outboard oscillator are:

Sound frequency.....	210 kc./sec.
Beam width.....	12°
Sounding rate.....	600 per minute
Theoretical accuracy.....	0.5 percent

When comparing profiles obtained by depth re-

corder with different oscillator specifications, some difference in results can be expected. The major difference will result from the resolving capabilities of the oscillator. Other factors being equal, the oscillator with the smallest angle of beam width will be more directional and will give a more detailed definition of the surface directly beneath the oscillator.

Distance wire and reel

The distance wire used is an 0.059-in.-diameter, steel piano wire. This wire is stored on a specially built, powered reel having a drum diameter of 9 in. (fig. 3.)

Measuring wheel

The distances across the lake are measured with a specially built measuring wheel having a circumference (at the center of the wire) of 2 ft. (fig. 4). An attached counter registers the revolutions of the wheel. A cam and micro-switch on the counter actuate a switch in the depth recorder to make a fix mark automatically on the depth-recorder scroll at intervals of 20 ft. The wire encircles the wheel to provide a maximum of friction and prevent slippage (fig. 5). The measuring wheel is mounted in the sounding boat directly over the depth-recorder oscillator. A light is mounted on the measuring wheel to serve as a target when, because of wind and rough water during the day, it becomes necessary to operate at night.

Radios

Transceiver radios having 0.75-w. transmitters are used for communication between the transitman, distance-reel operator, and boat operator. (fig. 6).

Wire splicer

A compressed-sleeve-type wire splicer is used for repairing breaks in the distance wire.

Method for Underwater Surveys

Stretching wire on range

A transit is set up on line at a point near one end of the range where the transitman has an unobstructed view of the range. The powered distance-wire reel is placed on range near the water's edge close to the transit. The loose end of the distance wire is then towed by boat across the lake and securely fastened at a point on range on the opposite bank. While the wire is being towed across the lake, radioed directions from the transitman guide the boat operator in keeping the boat on the range alinement. After the wire has been secured on the opposite bank, the wire is placed in an eye on a vertical post at the bow of the boat and the boat is then run back toward the transitman to place the wire on the exact range alinement. As the boat makes this return trip, the slack is taken up and

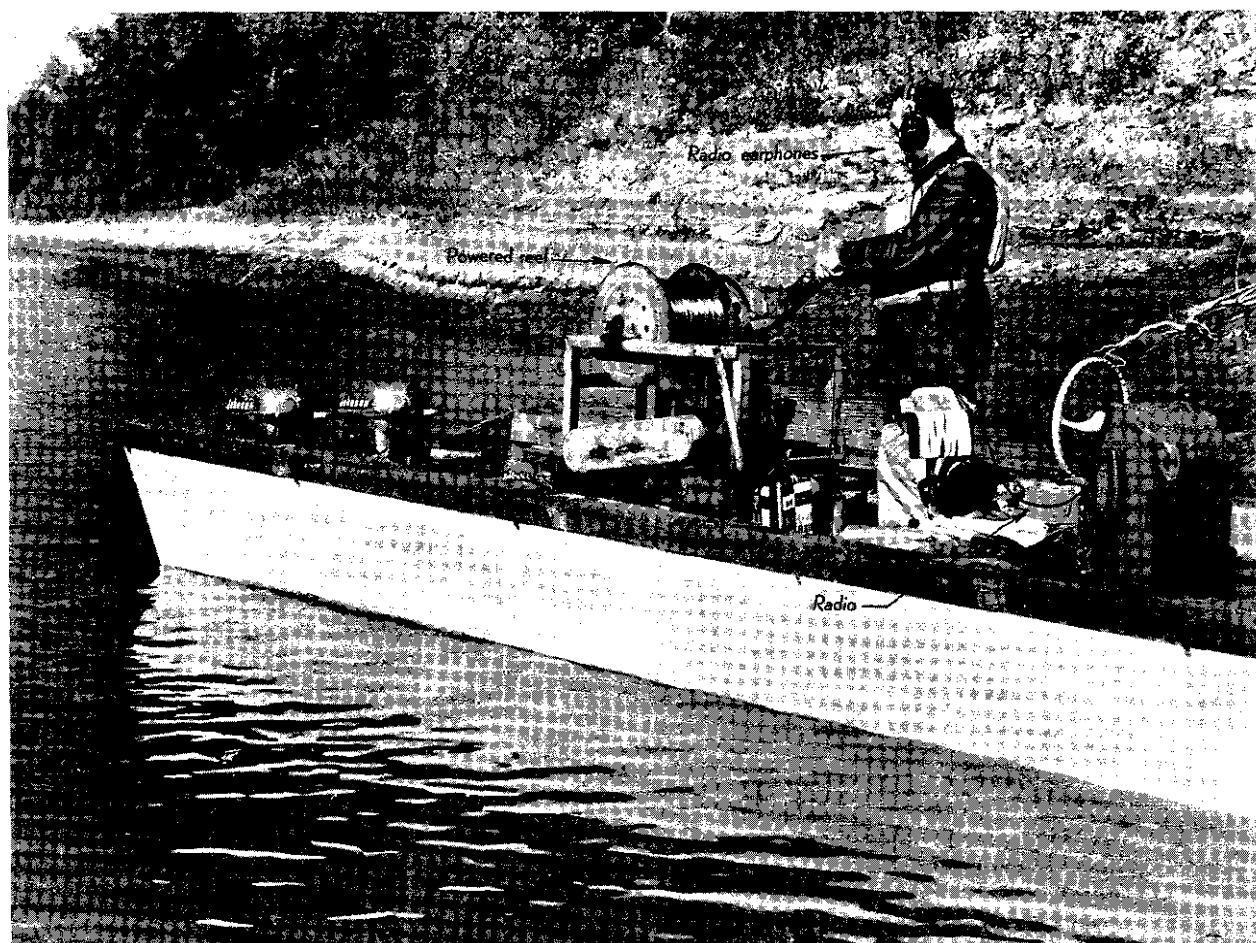


FIGURE 3. — Powered reel for piano wire.

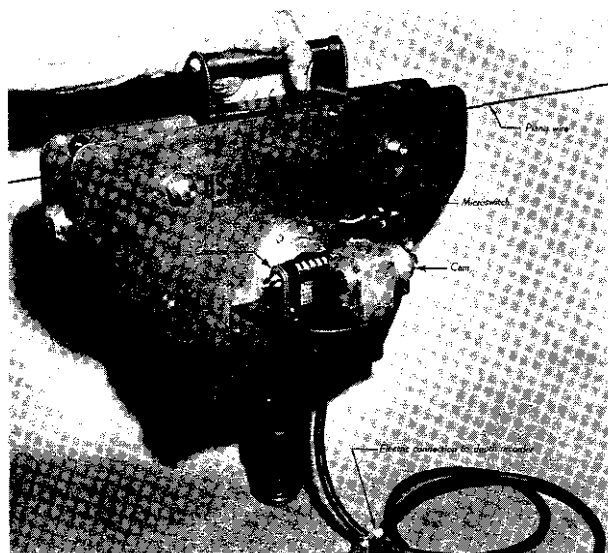


FIGURE 4. — Distance-measuring wheel.

buoys are fastened on the wire at approximate 300-ft. intervals. These floats holding up the wire reduce sag and the resulting errors in distance. Radioed alignment directions from the transitman to the boat operator are also given on this return trip. After the distance wire has been placed on range, the sounding boat is moved onto the range and the wire is threaded around the measuring wheel. The wire is then drawn taut and securely fastened at a point on the range.

Sounding the range

To establish a reference point for the sounding boat on the range stationing, the boat is securely held at a point on the distance wire and the range station at the measuring wheel is determined by a taped distance from a known station on the range (fig. 7). Before the sounding boat is moved from this point, the counter is set at zero and the cam is set under the microswitch to make the first fix mark on the

depth-recorder chart. Then as the boat moves forward on the range, the cam will strike the microswitch at 20-ft. intervals, automatically causing a fix mark to be made on the depth-

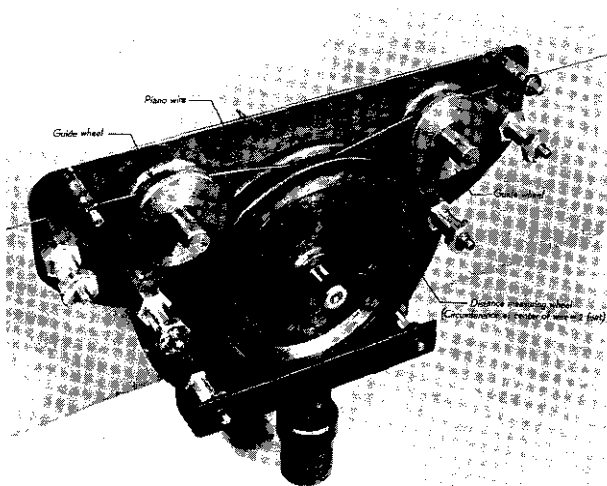


FIGURE 5. — Distance-measuring wheel with cover removed.



FIGURE 6. — Transitman at point on range.

recorder chart. The buoys on the wire furnish an approximate indication of alinement that enables the boat operator to evaluate the transitman's radioed directions better for maintaining a true course. On the sounding run, it is



FIGURE 7. — Sounding boat being zeroed on range.

necessary to remove and replace each buoy on the wire as the boat passes each buoy location. Figure 8 shows a sounding boat in operation on range. When the sounding boat reaches the opposite side of the reservoir, the counter reading represents the distance across the lake. The profile is extended from this point in the usual manner with levels and taped distances. Before the distance wire is recovered, the tension is released and the buoys are removed. The powered reel makes recovery of the wire more rapid and less laborious than by hand.

Results

Ranges up to 20,000 ft. have been profiled by this method. In accuracy tests on a 6,000-ft. taped course, the distances obtained by the distance-measuring wheel consistently agreed with the taped distances to the closest foot over the entire course and at numerous intermediate points. Under actual working conditions on ranges previously taped before the reservoir was filled, the measuring-wheel distances across the lake agreed with the original taped distances within ± 2 ft. on distances up to 13,600 ft.

Alinement errors can be held within any reasonable limits. Under actual working conditions, an experienced boat operator can, with care, keep the oscillator within 1 or 2 ft. of the

range alinement. This can be done without any appreciable loss of time.

In establishing and profiling the ranges by this method, it was considered necessary to conduct certain tests to determine if the depth-recorder survey would yield data that could be compared with direct-leveling surveys. Also, it was desirable to determine the effect of standing submerged timber in the reservoir pool upon the soundings.

To resolve these questionable points, several of the Blakely Mountain Reservoir ranges in the timbered areas were cleared to a width of 25 ft. These ranges were profiled by direct leveling before the reservoir was filled. Certain other ranges in the timbered areas were not cleared and were also profiled by direct leveling. These same ranges were resurveyed with the improved method after the reservoir was filled. Figure 9 is a chart comparing tests on one range.

A comparison of plotted results from surveys by both the direct leveling and improved methods indicates that:

- (1) In Arkabutla Reservoir in depths up to 23 ft., the soundings had an accuracy of ± 0.5 ft.
- (2) On test ranges in Blakely Mountain Reservoir where the original survey was

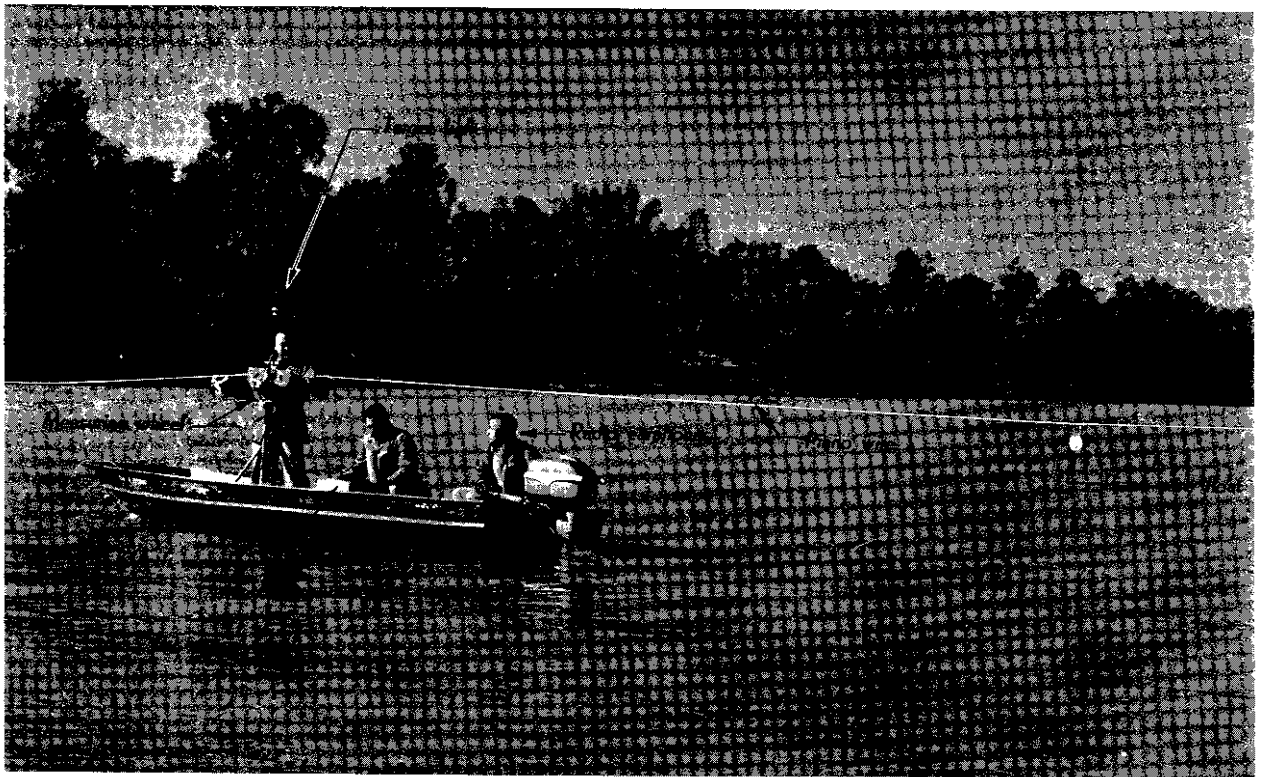


FIGURE 8. — Range sounding.

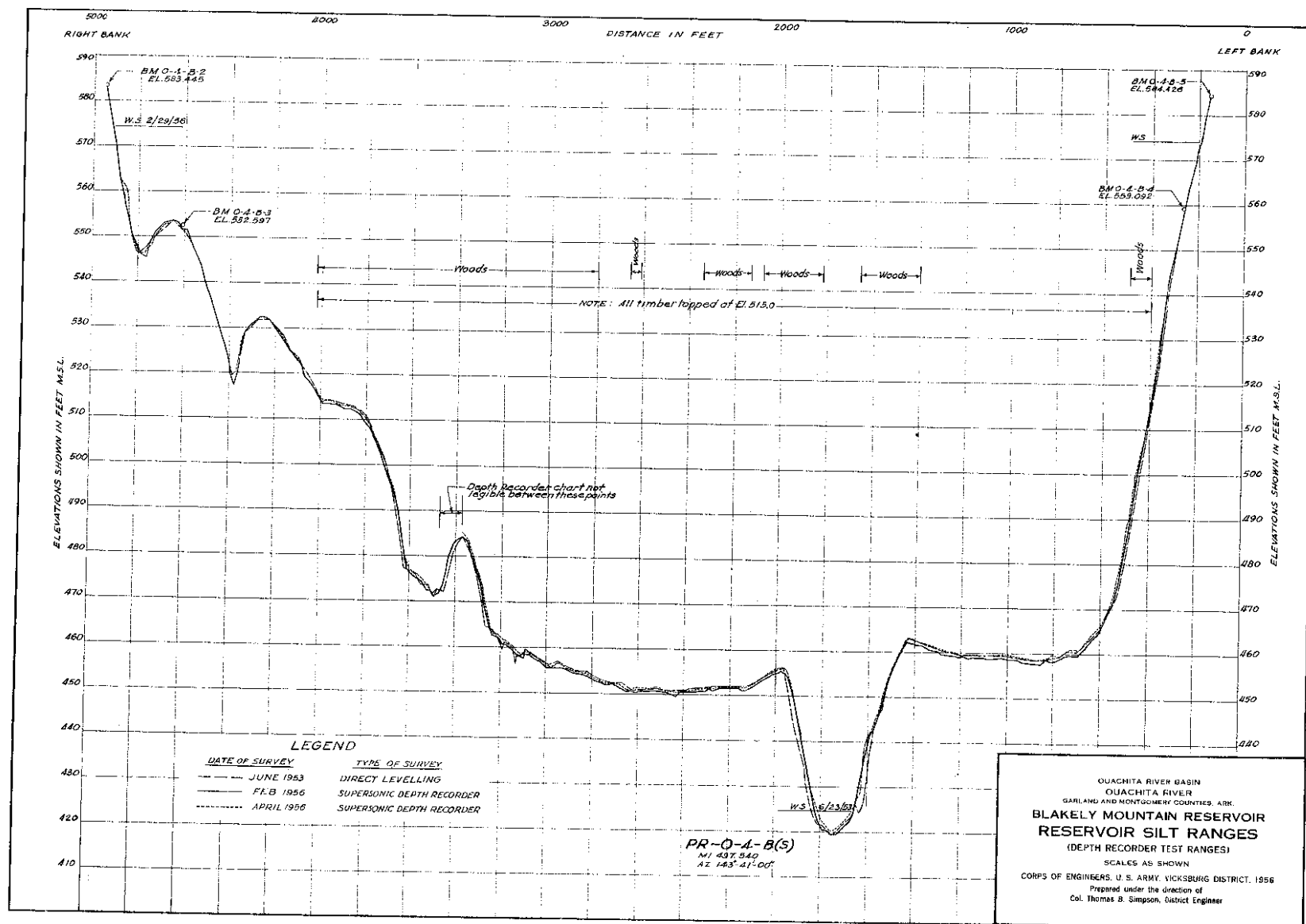


FIGURE 9. — Depth recorder test ranges, Blakeley Mountain Reservoir.

made by direct leveling and a check profile was run with the depth recorder, the soundings had a general accuracy of ± 1.0 ft. in depths up to 155 ft. on the valley bottom, flat shelves, and on relatively flat slopes. However, due to the mountainous terrain where steep underwater slopes were present on or immediately adjacent to the range, the depth-recorder soundings were influenced by echoes rebounding from these steep slopes and did not give the true depth directly underneath the sounding boat. Also, the depth recorder tends to smooth out sharp irregularities on the profile in deep water. These inaccuracies are inherent in the depth finder and are controlled by the beam width of the oscillator.

- (3) On test ranges in Blakely Mountain Reservoir, in check profiles obtained by the depth recorder over profiles previously surveyed by depth recorder, practically no differences were noted on the steep slopes; and the differences in the profiles on the valley bottom were ± 1.0 ft. in depths up to 180 ft.

Conclusions

The establishment of the original permanent-range system on Blakely Mountain Reservoir with the improved method was completed at a considerable saving in funds.

The improved method produces results well within the accuracy of other field methods available. This is especially true in deep water and on long ranges.

In terrain where steep underwater slopes are present, office adjustment will be required if the range profiles from this method are to be compared with direct-leveling profiles.

Good results are obtained when the original profile and the resurvey are both made with the improved method at comparable water stage. In these instances, the effect of echoes from steep slopes can be ignored, because these inaccuracies are relative and are present to the same extent in both profiles.

Comparison of the depth-recorder profiles with direct-leveling profiles on ranges in cleared and wooded areas did not indicate any appreciable effect of submerged, standing timber on the depth-recorder soundings.

This improved method can be successfully used for slack-water surveys on any reasonably calm body of water.

Continuing Improvements

The Vicksburg District is considering the development of a radar positioning instrument. This equipment, if feasible, will be used on Mississippi River hydrographic surveys. However, it may be useful in sedimentation surveys, especially in reservoirs having a high rate of silting.

This equipment will automatically track and plot the position of a sounding boat traveling at random over the area to be mapped. Position "fixes" on the map and depth "fixes" on the recorder chart will be made simultaneously by remote radio facilities. Therefore, depths from the recorder chart can be added to the plotted location of the boat at each "fixed" location. Depths and positions between "fixed" points can be interpolated.

The equipment's range is expected to be 2 miles when plotting at 1/10,000 scale. The expected accuracy within this range is ± 25 ft. The range of the equipment can be doubled to 4 miles with a resulting plotting scale of 1/20,000 and accuracy of ± 50 ft.

INITIAL UNIT WEIGHT OF DEPOSITED SEDIMENTS

[Paper No. 82]

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Introduction

In the course of conducting project investigation studies by the Bureau of Reclamation, it is frequently necessary to determine the average unit weight of sediments expected to accumulate in a reservoir. The storage capacity of a reservoir is progressively reduced with time, depending upon the rate of the inflowing sediments.

Estimating the average unit weight of these incoming sediments is necessary to determine the magnitude of the reduction in the reservoir capacity. The stream sediment transport load, generally measured in units of weight per time, can be converted to a volume quantity by means of the estimated unit weight.

It is the purpose of this paper to present a method of estimating the average initial unit weight of sediments deposited in a reservoir. The method developed is similar in principle to that of the Lane and Koelzer¹ studies and further investigated by Miller.²

¹ LANE, E. W., and KOELZER, V. A. DENSITY OF SEDIMENTS DEPOSITED IN RESERVOIRS. In *A Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams*. St. Paul U.S. Engin. Suboffice, Hydraul. Lab., Rpt. 9, 60 pp. 1943. Iowa Univ.

Data from many sources were compiled and analyzed by means of the IBM 650 electronic computer. A mathematical procedure of the method is presented by applying it to a specific example from a given set of field data.

Nomenclature

The term, "density," has often been used by sedimentologists to describe what is more appropriately known as unit weight or specific weight. This is apparently a carryover of the term used by Lane and Koelzer in the title of their report and used as a synonym for unit weight. However, density, as defined by Rouse,³ is expressed as *mass* per unit volume and designated as ρ (rho); Rouse further defines specific or unit weight as *weight* per unit volume expressed in pounds per cubic foot and has the symbol γ (gamma). In this paper the term, "unit weight," will be used.

In most sedimentation studies and investigations by the Bureau of Reclamation, the classification of sediments proposed by the American Geophysical Union is used. This classification, arranged according to type of sediments in the size ranges less than gravel, is indicated below:

<i>Sediment type</i>	<i>Size range (microns)</i>
Clay.....	Less than 4
Silt.....	4 to 62.5
Sand.....	62.5 to 2,000

Factors Affecting Unit Weight

There are several factors influencing the unit weight of sediments that deposit in a reservoir. The more dominant ones include: (1) The manner by which the reservoir is to be operated; (2) the texture and size of the sediment particles; (3) the compaction or consolidation rate of the deposited sediments; and (4) other less occurring or influential factors such as the action of density currents, thalweg slope of the incoming stream, and the effect existing vegetation in the reservoir headwaters area would have by screening out some of the incoming sediments.

Of the above-mentioned factors, probably the most influential is that of reservoir operation. Sediments depositing in a reservoir that is drawn down for considerable periods are sub-

jected to exposure. The sediments consequently dessicate and become more dense, thus increase the unit weight. On the other hand, reservoirs normally operating with a full pool would cause the unit weights to be lower, the deposited sediments not being exposed. Reservoirs operating under fluctuating levels impose an intermediate set of conditions that produce varying effects on the unit weight of the deposits, depending on where they are located in the reservoir.

The size of the incoming sediment particles has a significant effect upon unit weights. Sediments composed of high silts and sands would have higher unit weights than those in which clay predominates.

Compaction or consolidation rates are also of importance in affecting the unit weights of deposited sediments. Koelzer and Lara⁴ summarized the factors influencing these rates as (1) the weight of the overlying sediment, (2) the degree of exposure to drying, (3) particle size, (4) permeability, and (5) time. Little information is available as to the quantitative determination of the consolidation rates of the deposited sediments; therefore, they were excluded from the analysis presented in the development of the procedure in this paper.

Data

Sources of data used in the compilation (table 1) were numerous, although a major part of the data was obtained from reservoir survey reports by Federal and State agencies. A total of 1,316 samples were used, including those in the report of Lane and Koelzer.⁵

Table 1 lists information pertinent to each sample including (1) the location of the reservoir where the sample was taken, (2) observed unit weight, (3) percentages of clay, silt, and sand, (4) computed unit weight, and (5) the deviations of the computed from the observed unit weights for each sample.

The data have been arranged according to three types of reservoir operations and a fourth type that includes only river sediments. These types are described as follows:

- | | |
|-------------|--|
| <i>Type</i> | <i>Reservoir operation</i> |
| I..... | Sediment always submerged or nearly submerged |
| II..... | Normally moderate to considerable reservoir drawdown |
| III..... | Reservoir normally empty |
| IV..... | Riverbed sediments |

Selecting the proper type of reservoir operation from the above tabulation to apply to a specific problem is a matter of judgment. Information from the initial reservoir operation study relative to fluctuations in the reservoir level is a valuable aid in selecting the type of reservoir

² MILLER, C. R. DETERMINATION OF THE UNIT WEIGHT OF SEDIMENT FOR USE IN SEDIMENT VOLUME COMPUTATIONS. Bur. of Reclamation, 7 pp. 1953.

³ ROUSE, H. ELEMENTARY MECHANICS OF FLUIDS. 376 pp., illus. New York and London. 1946.

⁴ KOELZER, V. A., and LARA, J. M. DENSITY AND COMPACTION RATES OF DEPOSITED SEDIMENT. Amer. Soc. Civil Engin. Jour. of Hydraulics Div. 84(HY2): 1603-1 to 1603-15. 1958.

⁵ See footnote 1.

TABLE 1.—*Compilation of data for unit weight analysis*
 TYPE I. RESERVOIR OPERATION: SEDIMENT ALWAYS SUBMERGED OR NEARLY SUBMERGED

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Harry Strunk.....	Nebraska.....	USBR.....	1	<i>Lb./cu. ft.</i> 99.3	1.1	21.6	77.3	<i>Lb./cu. ft.</i> 90.35	8.95
			2	97.1	2.4	28.8	68.8	87.75	9.35
			3	87.9	2.5	10.6	62.0	85.86	2.04
			4	88.6	4.8	23.6	71.6	87.24	1.36
			5	80.7	8.2	77.0	14.8	70.53	10.17
			6	94.3	5.1	59.0	35.9	77.53	16.78
			7	72.1	10.2	86.5	3.3	66.41	5.69
			8	85.7	4.5	78.4	17.1	72.83	12.87
			9	84.3	4.6	73.9	21.5	73.24	10.53
			10	84.3	6.3	79.3	14.4	71.74	13.16
			11	91.4	4.5	68.5	27.0	75.53	15.87
			12	74.2	9.6	80.4	10.0	68.30	5.90
			13	79.6	3.9	77.2	18.9	73.37	6.23
			14	52.1	27.4	68.0	4.6	59.47	-7.37
			15	78.6	11.5	76.9	11.6	67.96	10.64
			16	61.4	9.8	71.4	18.8	70.73	-9.33
			17	45.7	36.4	61.0	2.6	54.97	-9.27
			18	39.9	16.2	75.6	8.2	65.12	-25.22
			19	30.0	60.7	38.0	1.3	43.43	-13.43
			20	47.7	25.9	70.6	3.5	59.64	-11.94
			21	60.4	11.8	83.6	4.6	66.07	-5.67
Leavenworth County State Lake.	Kansas.....	SCS.....	22	36.5	49.2	49.9	.9	48.71	-12.21
			23	58.0	25.6	72.4	2.0	59.10	-1.10
			24	72.0	23.2	68.8	8.0	62.04	9.96
			25	57.0	37.6	61.4	1.0	53.55	3.45
			26	73.0	20.0	77.0	3.0	62.01	10.99
			27	55.0	52.8	46.2	1.0	46.95	8.05
			28	70.0	39.2	53.8	7.0	54.73	15.27
			29	37.0	55.0	44.0	1.0	46.07	-9.07
			30	38.0	24.0	74.0	2.0	59.98	-21.98
			31	59.0	39.2	57.8	3.0	53.65	5.35
			32	47.5	39.4	60.5	.1	52.84	-5.34
Lake Calhoun.....	Illinois.....	SCS.....	33	68.0	24.9	74.7	.4	59.00	9.00
Lake Bracken.....	Illinois.....	SCS.....	34	48.5	38.9	60.0	1.1	53.11	-4.61
			35	39.4	41.4	58.0	.6	52.23	-12.83
Lancaster Reservoir..	South Carolina....	SCS.....	36	36.5	45.4	53.9	.7	50.47	-13.97
			37	53.6	61.4	37.6	1.0	43.43	10.17
			38	70.0	36.2	59.1	4.7	55.51	14.49
			39	70.2	20.0	67.0	13.0	64.71	5.49
			40	78.9	13.3	70.0	16.7	68.87	10.03
			41	68.4	24.8	68.2	7.0	60.89	7.51
			42	39.4	25.1	65.5	9.4	61.43	-22.03
			43	61.5	20.6	71.4	8.0	62.92	-1.42
Conchas Reservoir...	New Mexico.....	CE.....	44	20.7	67.0	32.0	1.0	40.79	-20.09
			45	84.0	4.0	14.0	82.0	90.38	-6.38
			46	64.2	19.0	80.0	1.0	61.91	2.29
			47	45.7	20.0	73.0	7.0	63.09	-17.39
			48	28.0	63.0	37.0		42.28	-14.28
			49	59.2	53.0	44.0	3.0	47.49	11.71
			50	40.5	40.0	55.0	5.0	53.75	-13.25
			51	35.1	51.0	43.0	6.0	49.18	-14.08
Tarryall Reservoir...	Colorado.....	FS.....	52	87.4	1.6	8.6	89.8	93.42	-6.02
			53	31.2	19.2	72.6	8.2	63.80	-32.60
Lake McBride.....	Iowa.....	SCS.....	54	81.1	4.8	16.8	78.4	88.86	-7.76
			55	59.0	22.0	77.0	1.0	60.59	-1.59
			56	59.0	20.0	77.0	3.0	62.01	-3.01
			57	62.0	17.0	81.0	2.0	63.06	-1.06
			58	56.0	16.0	80.0	4.0	64.04	-8.04
			59	60.0	14.0	85.0	1.0	64.11	-4.11
			60	60.0	22.4	22.1	54.5	74.90	-14.90
Lake El Dorado.....	Kansas.....	SCS.....	61	66.0	15.2	27.1	57.5	78.25	-12.25
Fort Supply Reservoir	Oklahoma.....	CE.....	62	41.3	56.0	39.0	5.0	46.71	-5.41
			63	35.5	72.0	24.0	4.0	39.40	-3.90
			64	33.5	77.0	20.0	3.0	36.93	-3.43
			65	30.5	79.0	19.0	2.0	35.78	-5.28
			66	63.9	32.0	26.0	42.0	67.26	-3.36
			67	37.5	68.0	30.0	2.0	40.62	-3.12

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Fort Supply Reservoir Continued	Oklahoma.....	CE.....	68	Lb./cu. ft. 33.5	74.0	25.0	1.0	Lb./cu. ft. 37.71	—4.21
			69	38.6	64.0	25.0	1.0	37.71	.89
			70	38.5	78.0	30.0	2.0	40.62	—2.12
			71	34.9	69.0	28.0	3.0	40.45	—5.55
			72	55.1	62.0	35.0	3.0	43.53	11.57
			73	37.5	75.0	22.0	3.0	37.81	—31
			74	58.5	53.0	36.0	11.0	49.65	8.85
			75	35.1	78.0	21.0	1.0	35.95	—85
			76	38.2	72.0	22.0	6.0	39.94	—1.74
			77	101.0	9.0	17.0	74.0	86.02	14.98
			78	58.2	51.0	19.0	30.0	55.66	2.45
			79	119.9	2.0	2.0	96.0	95.04	24.86
			80	73.8	47.0	26.0	27.0	56.61	17.19
			81	60.9	58.0	38.0	4.0	45.56	15.34
			82	117.5	3.0	1.0	96.0	94.60	22.90
			83	97.0	7.0	14.0	79.0	88.25	8.75
			84	80.9	28.0	38.0	34.0	66.86	14.04
			85	77.5	25.0	39.0	36.0	68.72	8.78
			86	98.4	2.0	3.0	95.0	94.77	3.63
Lake Texoma.....	Okla. and Texas...	CE.....	87	111.0	3.0	4.0	93.0	93.79	17.21
			88	41.0	62.0	37.0	1.0	42.99	—1.99
			89	44.0	52.0	46.0	2.0	47.66	—3.66
			90	48.0	41.0	56.0	3.0	52.77	—4.77
			91	64.0	44.0	48.0	8.0	52.80	11.20
			92	50.0	34.0	48.0	23.0	61.25	—11.25
			93	57.0	36.0	52.0	12.0	57.40	—40
			94	87.0	20.0	72.0	8.0	63.36	23.64
			95	69.0	30.0	47.0	23.0	63.01	5.99
			96	40.0	66.0	26.0	8.0	43.12	—3.12
			97	86.0	15.0	64.0	21.0	69.07	16.93
			98	56.0	42.0	49.0	9.0	53.95	2.05
			99	71.0	55.0	43.0	2.0	46.34	24.66
			100	76.0	13.0	67.0	20.0	69.68	6.32
			101	86.0	50.0	49.0	1.0	48.27	37.73
			102	74.0	20.0	75.0	5.0	62.55	11.45
			103	90.0	8.0	65.0	27.0	73.77	16.23
			104	83.0	9.0	65.0	25.0	72.79	10.21
			105	85.0	0	22.0	78.0	91.06	—6.06
			106	53.0	30.0	67.0	3.0	57.61	—4.61
			107	87.0	7.0	26.0	67.0	85.01	1.99
			108	79.0	28.0	64.0	8.0	59.84	19.16
			109	89.0	11.0	59.0	30.0	73.26	15.74
			110	74.0	12.0	60.0	28.0	72.28	1.72
			111	53.0	11.0	55.0	34.0	74.34	—21.34
			112	86.0	6.0	41.0	53.0	81.67	4.33
			113	89.0	8.0	61.0	31.0	74.85	14.15
			114	99.0	3.0	25.0	72.0	88.12	10.88
			115	98.0	3.0	3.0	94.0	94.06	3.94
			116	87.0	5.0	19.0	76.0	88.32	—1.32
			117	63.0	35.0	39.0	26.0	61.62	1.38
			118	53.0	36.0	49.0	15.0	58.21	—5.21
			119	35.0	73.0	27.0	0	37.88	—2.88
			120	86.0	11.0	36.0	53.0	79.47	6.53
			121	49.0	37.0	55.0	8.0	55.88	—6.88
			122	88.0	16.0	37.0	47.0	75.65	12.35
			123	39.0	74.0	25.0	1.0	37.71	1.29
			124	52.0	53.0	41.0	6.0	48.30	3.70
			125	49.0	56.0	47.0	7.0	47.25	1.75
			126	88.0	50.0	47.0	3.0	48.81	39.19
			127	88.0	12.0	64.0	24.0	71.20	16.80
			128	48.0	59.0	37.0	4.0	45.12	2.88
			129	82.0	30.0	46.0	24.0	63.28	18.72
			130	93.0	6.0	59.0	35.0	76.81	16.19
			131	58.0	45.0	35.0	20.0	55.60	2.40
			132	92.0	9.0	68.0	23.0	72.25	19.75
			133	44.0	60.0	28.0	2.0	44.14	—0.14
			134	91.0	26.0	71.0	3.0	59.37	31.63
			135	98.0	3.0	13.0	84.0	91.36	5.64

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Lake Texoma— Continued	Okla. and Texas...	CE.....	136	<i>Lb./cu. ft.</i> 79.0	20.0	25.0	55.0	<i>Lb./cu. ft.</i> 76.05	2.95
			137	100.0	4.0	26.0	70.0	87.14	12.86
			138	89.0	3.0	37.0	60.0	84.88	4.12
			139	91.0	5.0	25.0	70.0	86.70	4.30
			140	97.0	3.0	22.0	57.0	88.93	8.07
			141	89.0	6.0	37.0	57.0	82.75	6.25
			142	81.0	7.0	43.0	50.0	80.42	0.58
			143	87.0	3.0	17.0	80.0	90.28	-3.28
			144	96.0	3.0	1.0	96.0	94.70	1.40
			145	96.0	2.0	3.0	95.0	94.77	1.23
			146	108.0	3.0	1.0	96.0	94.60	13.40
			147	74.26	60.0	23.0	17.0	48.19	26.07
			148	84.24	74.0	15.0	11.0	40.41	43.83
			149	41.81	86.0	13.0	1.0	32.43	9.38
Lake Bloomington....	Illinois.....	Ill. State Water Survey.	150	42.43	87.0	12.0	1.0	31.99	10.44
			151	42.43	88.0	11.0	1.0	31.55	10.88
			152	36.19	89.0	9.0	2.0	31.38	4.81
			153	41.18	81.0	12.0	7.0	36.25	4.93
			154	27.46	86.0	11.0	3.0	32.97	-5.51
			155	28.08	92.0	7.0	1.0	29.79	-1.71
			156	29.33	86.0	11.0	3.0	32.97	-3.64
			157	32.45	91.0	6.0	3.0	30.77	1.68
			158	64.90	65.0	17.0	18.0	46.26	18.64
			159	56.78	77.0	20.0	3.0	36.93	19.85
			160	45.55	77.0	22.0	1.0	36.39	9.16
			161	49.30	81.0	18.0	1.0	34.63	14.67
			162	41.81	85.0	14.0	1.0	32.87	8.94
Lake Decatur.....	Illinois.....	SCS.....	163	39.94	43.4	38.6	18.0	55.94	-16.00
			164	34.32	66.2	32.3	1.5	41.50	-7.18
			165	24.96	67.8	32.0	.2	40.08	-15.12
			166	24.96	35.5	58.9	5.6	55.78	-30.82
			167	37.44	57.7	39.4	2.9	45.29	-7.85
			168	37.44	56.5	42.7	.8	45.63	-8.19
			169	36.82	65.0	34.6	.4	41.40	-4.58
			170	24.34	56.2	43.2	.6	45.63	-21.29
			171	19.97	57.8	41.4	.8	44.75	-24.78
			172	33.07	63.3	36.5	.2	42.28	-9.21
			173	33.07	46.8	52.8	.4	49.32	-16.25
			174	33.07	66.7	33.0	.3	40.52	-7.45
			175	30.58	73.4	26.4	.2	37.88	-7.30
			176	29.33	76.9	23.0	.1	36.12	-6.79
			177	21.84	58.3	40.9	.8	44.75	-22.91
			178	36.82	54.1	45.7	.2	46.24	-9.42
			179	39.31	55.3	44.5	.2	45.80	-6.49
			180	42.43	56.9	43.1	.0	44.92	-2.49
			181	38.69	51.3	48.6	.1	47.56	-8.87
			182	35.57	54.7	41.4	3.9	46.88	-11.31
			183	38.69	62.2	37.0	.8	42.99	-4.30
			184	41.81	59.3	38.7	2.0	44.58	-2.77
			185	38.06	54.7	44.5	.8	46.07	-8.01
			186	34.94	64.4	34.9	.7	42.11	-7.17
			187	38.06	54.3	45.5	.2	46.24	-8.18
			188	42.43	61.4	38.5	.1	43.16	-0.73
			189	35.57	57.8	42.1	.1	44.48	-8.91
			190	41.81	51.8	48.1	.1	47.12	-5.31
			191	33.07	50.7	44.1	.2	47.56	-14.49
			192	31.20	47.8	52.1	.1	48.88	-17.68
			193	37.44	52.3	47.5	.2	47.12	-9.68
			194	41.81	43.6	56.2	.2	50.64	-8.83
			195	48.67	47.4	52.5	.1	49.32	-.65
			196	48.67	43.8	56.1	.1	50.64	-1.97
			197	51.79	47.1	52.8	.1	49.32	2.47
			198	49.30	48.0	51.7	.3	48.88	-.42
			199	53.04	47.2	52.7	.1	49.32	3.72
			200	49.92	45.3	54.5	.2	50.20	-.28
			201	56.78	45.5	54.0	.5	50.20	6.58
			202	70.51	29.1	67.4	3.5	58.32	12.19
			203	70.51	11.8	20.7	67.5	82.81	-12.30

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight			
					Clay	Silt	Sand					
Lake Decatur-- Continued	Illinois.....	SCS.....	204	Lb./cu. ft. 68.02	32.4	66.9	.7	L/cu. ft. 56.19	11.83			
			205	64.27	30.1	69.4	.5	56.80	7.47			
			206	56.16	36.9	56.1	7.0	55.61	.55			
			207	62.40	33.0	65.5	1.5	55.75	6.65			
			208	31.82	66.1	33.7	.2	40.96	-9.14			
			209	36.82	59.5	40.0	.5	44.04	-7.22			
			210	43.06	33.4	62.7	3.9	56.56	-13.50			
			211	44.30	47.3	52.3	.4	49.32	-5.02			
			212	31.20	67.2	32.3	.5	40.52	-9.32			
			213	34.32	55.8	43.9	.3	45.36	-11.04			
			214	36.82	61.1	38.6	.3	43.16	-6.34			
			215	38.69	41.2	58.7	.1	51.96	-13.27			
			216	63.65	30.4	66.2	3.4	57.61	6.04			
			217	44.93	42.3	55.5	2.2	52.06	-7.13			
			218	51.79	43.1	56.7	.2	51.08	.71			
			219	43.68	39.8	59.6	.6	52.67	-8.99			
			220	49.92	42.9	56.7	.4	51.08	-1.16			
			La Von Reservoir.....	Texas.....	CE.....	221	54.3	43.0	48.2	8.8	53.51	.79
						222	36.2	69.0	27.9	3.1	40.45	-4.25
						223	29.4	85.5	14.3	.2	32.60	-3.20
224	46.9	67.0				32.2	.8	40.79	6.12			
225	55.6	55.2				42.9	1.9	46.34	9.26			
226	31.5	80.0				19.1	.9	35.07	-3.57			
227	34.7	83.0				16.7	.3	33.48	1.22			
228	45.2	63.0				35.7	1.3	42.55	2.65			
229	39.2	69.5				29.2	1.3	39.91	-.71			
230	46.9	54.0				33.8	12.2	49.48	-2.58			
231	69.1	70.6				28.5	.9	39.03	30.07			
232	60.0	61.0				37.8	1.2	43.43	16.57			
233	25.4	78.4				19.8	1.8	36.22	-11.72			
234	30.9	70.8				23.0	6.2	40.38	-9.48			
235	36.9	61.2				32.7	6.1	44.78	-7.88			
236	22.0	73.8				24.1	2.1	37.98	-15.98			
237	65.2	53.5				45.4	1.1	46.95	18.25			
238	52.4	53.8				44.6	1.6	46.78	5.62			
239	69.6	47.0				37.2	15.8	53.64	15.96			
Hulah Reservoir.....	Oklahoma.....	CE.....				240	73.5	40.0	45.5	14.5	56.18	17.32
			241	57.4	41.0	48.2	10.8	54.93	2.47			
			242	36.9	65.5	33.1	1.4	41.67	-4.77			
			243	37.0	77.8	21.7	.5	35.68	1.32			
			244	56.5	47.5	44.9	7.6	51.48	4.92			
			245	72.0	69.4	29.7	.9	39.91	32.09			
			246	70.5	63.5	33.1	3.4	43.09	27.41			
			247	60.0	75.0	24.1	.9	37.27	22.73			
			248	72.0	70.0	29.1	.9	29.47	32.53			
			249	60.5	63.8	29.5	6.7	43.73	16.77			
			250	50.7	48.0	40.0	12.0	52.12	-1.42			
			251	37.6	62.0	37.0	1.0	42.99	-5.39			
			252	52.8	38.0	59.0	3.0	53.28	-.48			
			253	42.1	63.0	34.0	3.0	43.09	-.99			
			254	40.7	52.0	45.0	3.0	47.93	-7.23			
			255	41.5	58.0	37.0	5.0	45.83	-4.33			
			256	40.4	56.0	40.0	4.0	46.44	-6.04			
			257	36.5	72.0	25.0	3.0	39.13	-2.63			
			258	40.7	64.0	35.0	1.0	42.11	-1.41			
			TYPE II. -- RESERVOIR OPERATION: NORMALLY A MODERATE TO CONSIDERABLE RESERVOIR DRAWDOWN	Berlin Reservoir.....	Pittsburg District.	CE.....	1	63.4	60.0	38.0	2.0	49.92
2	73.4	50.0					44.0	6.0	54.56	18.84		
3	78.9	34.0					61.0	5.0	60.06	18.84		
4	69.0	19.0					43.0	38.0	74.04	-5.04		
5	43.3	21.0					52.0	27.0	70.46	-27.16		
6	82.8	25.0					69.0	6.0	63.56	19.24		

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Berlin Reservoir— Continued	Pittsburg District.	CE.....	7	<i>Lb./cu. ft.</i> 76.4	2.0	7.0	91.0	<i>Lb./cu. ft.</i> 93.94	-17.54
			8	48.5	12.0	33.0	55.0	80.98	-32.48
			9	71.0	47.0	39.0	14.0	57.72	13.28
Seminole Reservoir....	Wyoming.....	USBR.....	10	45.6	61.0	38.9	.1	49.04	-3.44
			11	34.8	55.0	43.5	1.5	51.59	-16.79
			12	44.8	54.5	41.0	4.5	52.55	-7.75
Elephant Butte Reservoir.	New Mexico.....	USBR.....	13	68.1	20.8	51.8	27.4	70.46	-2.36
			14	87.8	10.4	39.2	50.4	80.40	7.40
			15	85.7	4.0	34.3	61.7	85.68	.02
			16	76.0	44.0	47.6	8.4	57.24	18.76
			17	75.8	43.2	47.4	9.4	57.86	17.94
			18	69.9	70.4	27.6	2.0	46.32	23.58
			19	66.6	57.6	36.9	5.5	51.69	14.91
			20	90.0	19.2	15.8	65.0	81.06	8.94
			21	66.8	60.8	33.5	5.7	50.60	16.20
			22	80.7	22.4	52.2	25.4	69.58	11.12
			23	88.5	9.6	69.0	21.4	72.86	15.64
			24	93.8	8.0	65.3	26.7	75.14	18.66
			25	76.9	45.6	45.6	8.8	56.78	20.12
			26	76.7	26.4	61.6	12.0	64.76	11.94
			27	66.2	71.2	26.1	2.7	46.22	19.98
			28	62.4	72.8	24.0	3.2	45.50	16.90
			29	81.6	17.6	31.9	50.5	77.79	3.81
			30	64.8	68.8	27.2	4.0	47.20	17.60
			31	58.0	65.6	29.7	4.7	48.54	9.46
			32	73.5	31.2	14.5	54.3	73.88	.38
			33	87.7	17.6	25.2	57.2	79.34	8.36
			34	82.5	29.6	30.7	39.7	70.60	11.90
			35	85.0	10.4	60.9	28.7	74.94	10.06
			36	88.8	6.4	17.4	76.2	68.60	.20
			37	86.4	5.6	32.2	62.2	84.96	1.44
			38	72.6	36.0	43.1	20.9	63.50	9.10
			39	66.7	43.2	47.4	9.4	57.86	8.84
			40	71.1	32.0	55.5	12.5	62.73	8.37
			41	52.7	48.0	44.5	7.5	55.67	-2.97
			42	82.1	12.0	35.5	52.5	80.33	1.77
			43	35.7	64.0	31.8	4.2	49.00	-13.30
			44	35.3	60.8	34.4	4.8	50.34	-15.04
			45	28.5	72.0	26.3	1.7	45.60	-17.10
			46	30.9	73.6	25.1	1.3	44.62	-13.72
			47	76.8	13.5	50.1	36.4	75.60	1.20
			48	100.0	1.6	6.2	92.2	94.20	5.80
			49	79.3	18.4	50.4	31.2	72.58	6.72
			50	66.2	21.6	57.0	21.4	68.54	-2.34
			51	40.9	68.0	29.5	2.5	47.17	-6.27
			52	39.0	75.2	23.8	1.0	44.26	-5.26
			53	16.3	66.4	30.8	2.8	48.02	-31.72
			54	29.9	69.6	28.2	2.2	46.32	16.42
			55	40.9	72.0	26.3	1.7	45.60	-4.70
			56	34.4	71.2	27.0	1.8	45.96	-11.56
			57	31.8	52.0	42.2	5.8	53.84	-22.04
			58	35.5	63.2	33.3	3.5	49.16	-13.66
			59	40.1	72.8	25.7	1.5	45.18	-5.08
			60	28.3	58.4	37.1	4.5	51.15	-22.85
			61	34.7	71.2	27.0	1.8	45.96	-11.26
			62	40.2	68.0	29.5	2.5	47.17	-6.97
			63	38.8	74.4	24.4	1.2	44.62	-5.82
			64	32.9	68.8	28.9	2.3	46.68	-13.78
			65	39.7	71.2	27.0	1.8	45.96	-6.26
			66	24.8	68.0	29.5	2.5	47.17	-22.37
			67	31.6	76.8	22.5	.7	43.54	-11.94
			68	39.0	73.6	25.1	1.3	44.62	-5.62
			69	19.6	65.6	31.4	3.0	48.02	-28.42
			70	29.3	75.2	23.8	1.0	44.26	-14.96
			71	39.6	73.6	25.1	1.3	44.62	-5.02
			72	19.6	69.6	28.2	2.2	46.32	-26.72
			73	29.1	75.2	23.8	1.0	44.26	-15.16
			74	38.2	72.0	26.3	1.7	45.60	-7.40

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Elephant Butte— Continued	New Mexico.....	USBR.....		<i>Lb./cu. ft.</i>				<i>Lb./cu. ft.</i>	
			75	18.8	69.6	28.2	2.2	46.32	-27.52
			76	29.9	72.8	25.7	1.5	45.18	-15.28
			77	36.5	74.4	24.4	1.2	44.62	-8.12
			78	30.4	70.4	27.6	2.0	46.32	-15.92
			79	36.1	61.6	34.6	3.8	49.72	-13.62
			80	41.0	72.0	26.3	1.7	45.60	-4.60
			81	25.8	72.8	25.7	1.5	45.18	-19.38
			82	34.3	73.6	25.1	1.3	44.62	-10.32
			83	41.0	73.6	25.1	1.3	44.62	-3.62
			84	29.6	76.0	23.2	.8	43.90	-14.30
			85	35.9	58.4	33.8	7.8	52.20	-16.30
			86	42.7	70.4	27.6	2.0	46.32	-3.62
			87	39.0	69.6	23.2	2.2	46.32	-7.32
			88	67.28	48.5	51.3	.2	53.72	13.56
			89	76.63	17.0	47.2	35.8	74.24	2.39
			90	64.17	32.8	66.7	.5	59.32	4.85
			91	72.27	17.5	80.3	2.2	65.27	7.00
			92	64.17	77.0	21.4	1.6	43.80	20.37
			93	71.65	74.5	25.3	.2	44.23	27.42
			94	70.40	82.5	17.1	.4	41.40	29.00
			95	82.86	11.6	59.7	28.7	74.22	8.64
			96	74.14	84.3	15.5	.2	40.76	33.38
			97	58.56	75.1	24.5	.4	44.00	14.56
			98	72.27	34.7	35.6	29.7	66.20	6.07
			99	79.12	20.7	65.9	13.4	66.82	12.30
			100	77.25	67.2	32.3	.5	46.94	30.31
			101	87.84	5.4	51.4	43.2	80.38	7.46
			102	87.84	3.1	23.1	73.8	89.16	-1.32
			103	74.14	18.1	73.3	8.6	66.86	7.28
			104	85.97	14.1	18.8	67.1	83.38	2.59
			105	37.90	58.0	40.0	2.0	50.64	-12.74
			106	37.46	71.0	27.5	1.5	45.83	-8.37
			107	35.69	82.0	18.0	0	41.48	-5.79
			108	34.07	76.0	24.0	0	43.64	-9.57
			109	33.44	83.0	17.0	0	41.12	-7.68
			110	29.50	69.0	29.0	2.0	46.68	-17.18
			111	33.96	70.0	29.0	1.0	46.06	-12.10
			112	36.23	83.0	17.0	0	41.12	-4.89
			113	33.03	85.0	14.0	1.0	40.66	-7.63
			114	34.07	78.0	21.0	1.0	43.18	-9.11
			115	31.76	81.0	19.0	0	41.84	-10.08
			116	36.54	76.0	23.0	1.0	43.90	-7.36
			117	35.39	81.0	19.0	0	41.84	-6.45
			118	42.50	65.0	35.0	0	47.60	-5.10
			119	33.67	84.0	16.0	0	40.76	-7.09
			120	38.62	77.0	22.0	1.0	43.54	-4.92
			121	54.80	75.0	25.0	0	44.00	10.80
			122	65.41	79.0	21.0	0	42.56	22.85
			123	67.48	66.0	32.0	2.0	47.76	19.72
			124	77.40	21.0	43.0	36.0	72.80	4.60
			125	38.40	79.0	21.0	0	42.56	-4.16
			126	30.24	80.0	19.0	1.0	42.46	-12.22
Lake Carthage.....	Illinois.....	Ill. State Water Survey.	127	60.5	23.0	76.0	1.0	62.98	-2.48
			128	68.0	20.2	78.4	1.4	64.06	3.94
			129	46.8	30.9	69.0	.1	59.84	-13.40
			130	46.2	35.2	64.7	.1	58.40	-12.20
			131	45.6	35.8	63.8	.4	58.04	-12.44
			132	40.6	43.2	56.7	.1	55.52	-14.92
			133	44.3	40.5	59.4	.1	56.45	-12.15
			134	41.2	47.8	52.1	.1	53.72	-12.52
			135	46.8	39.9	59.8	.3	56.60	-9.80
			136	41.8	24.2	75.7	.1	62.36	-20.56
Altus Reservoir.....	Oklahoma.....	USBR.....	137	60.3	21.4	75.2	3.4	64.22	-3.92
			138	26.3	19.3	66.2	14.5	67.82	-41.52
			139	26.7	38.7	59.1	2.2	57.48	-30.78
			140	53.6	24.2	50.9	24.9	68.86	-15.26
			141	36.5	32.8	57.0	10.2	61.72	-25.22
			142	91.6	18.3	21.9	59.8	80.12	11.48

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Altus Reservoir— Continued	Oklahoma.....	USBR.....	143	<i>Lb./cu. ft.</i> 106.6	1.1	10.2	88.7	<i>Lb./cu. ft.</i> 93.78	12.82
			144	45.8	36.8	62.4	.8	57.94	-12.14
			145	65.4	4.6	45.3	50.1	82.20	-16.80
			146	80.8	21.1	44.4	34.5	72.37	8.43
			147	54.3	49.2	48.5	2.3	53.88	.42
			148	57.9	20.2	58.0	21.8	69.52	-11.62
			149	32.8	31.6	53.1	15.3	63.38	-30.58
			150	39.6	47.1	51.7	1.2	54.34	-14.74
			151	43.1	46.9	51.1	2.0	54.60	-11.50
			152	49.3	48.9	50.3	.8	53.62	-4.32
			153	36.1	44.0	52.9	3.1	55.94	-19.84
			154	48.6	18.4	50.1	31.5	72.57	-23.97
			155	51.9	16.0	64.7	19.3	70.18	-18.28
			156	56.3	27.0	44.7	28.3	68.56	-12.26
			157	38.3	18.0	69.9	12.1	67.64	-29.34
			158	49.8	21.0	36.4	42.6	74.62	-24.82
			159	19.9	53.0	42.6	4.4	52.96	-33.06
			160	19.3	47.4	46.0	6.6	55.90	-36.60
			161	79.8	5.8	48.7	45.5	80.74	-.94
			162	62.2	36.5	24.6	38.9	67.97	5.77
			163	82.6	9.3	61.7	29.0	75.30	7.30
			164	63.2	16.6	15.3	68.1	82.56	-19.36
			165	88.1	15.0	29.8	55.2	79.90	8.20
			166	63.5	53.0	40.5	6.5	53.61	9.89
			167	61.5	59.0	35.4	5.6	51.32	10.18
			168	101.7	3.8	15.3	80.9	90.62	11.08
			169	69.9	36.4	50.8	12.8	61.42	8.48
			170	78.4	32.0	35.6	32.4	67.80	10.60
			171	82.3	50.8	37.1	12.1	55.76	26.54
			172	97.8	12.8	14.0	73.2	85.30	12.50
			173	95.1	0	2.8	97.2	96.22	-1.12
			174	82.3	13.0	35.2	51.8	79.84	2.46
			175	88.5	3.8	19.9	76.3	89.32	-.82
			176	88.5	5.7	19.2	75.1	88.34	.16
Lake Brownwood....	Texas.....	Water Improvement District No. 1.	177	39.6	60.8	25.2	14.0	52.68	-13.08
			178	32.7	57.6	35.2	7.2	51.94	-19.24
			179	22.2	67.2	23.5	9.3	49.22	-27.02
			180	42.0	66.4	24.9	8.7	49.58	-7.58
			181	33.8	57.6	29.4	13.0	53.50	-19.70
			182	39.8	66.4	24.1	9.5	49.57	-9.77
			183	42.9	59.2	32.3	8.5	51.90	-9.00
			184	41.1	62.4	28.9	8.7	51.02	-9.92
			185	49.3	56.0	32.3	11.7	53.96	-4.66
			186	50.2	48.0	37.0	15.0	57.62	-7.42
			187	32.7	71.2	21.1	7.7	47.52	-14.82
			188	36.4	66.4	24.9	8.7	49.58	-13.18
			189	38.4	68.8	27.2	4.0	47.20	-8.80
			190	43.8	62.4	25.6	12.0	51.80	-8.00
			191	40.2	67.2	24.3	8.5	49.02	-8.82
			192	35.6	72.0	24.7	3.3	45.86	-10.26
			193	33.9	67.2	23.5	9.3	49.22	-10.32
			194	42.2	59.2	30.6	10.2	52.36	-10.16
			195	50.9	48.0	37.0	15.0	57.62	-6.72
			196	48.4	45.6	39.7	14.7	58.34	-9.94
			197	45.6	56.0	30.7	13.3	54.22	-8.62
			198	33.6	68.8	20.2	11.0	49.02	-15.42
Angostura Reservoir..	South Dakota.....	USBR.....	199	51.1	45.6	33.9	20.5	59.91	-8.81
			200	41.9	55.9	44.1	0	50.84	-8.94
			201	38.5	56.5	43.5	0	50.66	-12.16
			202	45.3	53.9	46.1	0	51.56	-6.26
			203	85.6	8.9	33.4	57.7	82.84	2.76
			204	86.9	10.3	57.1	32.6	75.98	10.92
			205	34.6	76.3	23.7	0	43.64	-9.04
Guernsey Reservoir...	Wyoming.....	USBR.....	206	37.0	70.9	29.1	0	45.44	-8.44
			207	36.0	71.0	29.0	0	45.44	-9.44
			208	31.8	71.1	28.9	0	45.44	-13.64
			209	40.8	73.0	27.0	0	44.72	-3.92
			210	48.6	68.0	32.0	0	46.52	2.08

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt. <i>Lb./cu. ft.</i>	Percent			Comp. unit wt. <i>Lb./cu. ft.</i>	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Guernsey Reservoir— Continued	Wyoming	USBR	211	59.6	41.0	55.0	4.0	57.28	2.32
			212	71.6	29.0	60.0	11.0	63.42	8.18
			213	70.0	41.0	55.0	4.0	57.28	12.72
			214	74.4	29.0	64.0	7.0	62.38	12.02
			215	53.9	44.0	53.0	3.0	55.94	-2.04
			216	82.3	39.0	56.0	5.0	58.26	24.04
Morena Reservoir	California	City of San Diego . .	217	76.9	0	15.0	85.0	93.10	-16.20
			218	93.8	0	3.0	97.0	96.22	-2.42
			219	72.5	0	22.0	78.0	91.28	-18.78
			220	69.4	0	37.0	63.0	87.38	-17.98
			221	84.4	0	13.0	87.0	93.62	-9.22
			222	95.0	2.0	2.0	96.0	95.24	-.24
			223	96.3	3.0	1.0	96.0	94.88	1.42
			224	71.9	1.0	6.0	93.0	94.82	-22.92
			225	83.1	0	14.0	86.0	93.86	-10.26
			226	69.4	0	39.0	61.0	86.86	-17.46
			227	93.1	0	1.0	99.0	96.74	-3.64
			228	91.3	0	25.0	75.0	90.50	.80
			229	81.3	0	7.0	93.0	95.18	-13.88
			230	90.0	0	0.0	100.0	97.00	-7.00
			231	54.0	35.0	60.0	5.0	59.70	-5.70
Fern Ridge Reservoir.	Oregon	CE	232	56.0	32.0	62.0	6.0	61.04	-5.04
Arrowrock Reservoir.	Idaho	USBR	233	53.9	13.0	54.0	33.0	74.90	-21.00
			234	87.9	1.0	7.0	92.0	94.56	-6.66
			235	60.4	6.0	24.0	70.0	87.04	-26.64
			236	44.2	41.0	49.0	10.0	57.84	-13.64
			237	102.2	1.0	4.0	95.0	94.34	7.66
			238	64.2	13.0	26.0	61.0	82.18	-17.98
			239	52.7	25.0	54.0	21.0	67.46	-14.76
			240	61.5	13.0	68.0	19.0	71.26	-9.76
			241	85.2	1.0	2.0	97.0	95.86	-10.66
			242	47.7	10.0	56.0	34.0	76.24	28.54
			243	57.9	11.0	61.0	28.0	74.32	-16.42
			244	52.3	14.0	60.0	26.0	72.72	-20.42
			245	58.8	10.0	46.0	44.0	78.84	-20.04
			246	57.3	10.0	60.0	30.0	75.20	-17.90
			247	48.3	10.0	56.0	34.0	76.24	-27.94
			248	112.5	1.0	4.0	95.0	95.34	17.16
			249	85.9	0	4.0	96.0	95.96	-10.06
			250	52.1	26.0	53.0	21.0	67.10	-15.00
Lake Corpus Christi.	Texas	SCS	251	34.0	77.0	19.0	4.0	44.32	-10.32
			252	55.0	68.0	27.0	5.0	47.82	7.18
			253	62.0	39.0	53.0	8.0	59.04	2.96
			254	30.0	87.0	7.0	6.0	41.24	-11.24
			255	33.8	78.0	17.0	5.0	44.22	-10.42
			256	57.8	62.0	23.0	15.0	52.58	5.22
Lake Waco	Texas	SCS	257	42.2	76.0	21.0	3.0	44.42	-2.22
			258	55.2	54.0	32.0	14.0	55.20	0
			259	49.3	60.0	26.0	14.0	53.04	-3.74
			260	42.2	80.0	16.0	4.0	43.24	-1.04
			261	45.6	70.0	22.0	8.0	47.88	-2.28
			262	70.1	57.0	23.0	20.0	55.68	14.42
			263	73.7	32.0	17.0	51.0	72.74	.96
			264	75.9	50.0	27.0	23.0	58.98	16.92
			265	64.0	18.0	71.0	11.0	67.38	-3.38
Cold Springs Reservoir.	Oregon	SCS	266	70.0	53.0	31.0	16.0	56.08	13.92
			267	77.0	15.0	31.0	54.0	79.64	-2.64
			268	66.0	18.0	66.0	16.0	68.68	-2.68
			269	72.0	14.0	42.0	44.0	77.40	-5.40
			270	82.0	14.0	47.0	39.0	76.10	5.90
			271	92.0	14.0	52.0	34.0	74.80	17.20
			272	101.0	11.0	8.0	81.0	88.10	12.90
			273	72.0	18.8	47.7	33.5	72.94	-.94
			274	78.5	24.6	45.6	29.8	69.80	8.70
			275	80.6	20.8	46.0	33.2	72.02	8.58
Tongue River Reservoir.	Montana	USBR	276	42.6	49.6	49.0	1.4	53.26	-10.66
			277	43.6	63.0	35.5	1.5	48.71	-5.11
			278	64.9	18.9	65.7	15.4	68.06	-3.16

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Tongue River Reservoir—Con.	Montana.....	USBR.....		<i>Lb./cu. ft.</i>				<i>Lb./cu. ft.</i>	
			279	74.5	31.5	40.5	28.0	66.94	7.56
			280	83.0	21.6	49.3	29.1	70.62	12.38
			281	37.2	52.1	45.5	2.4	52.80	-15.60
			282	41.5	58.4	40.8	0.8	50.38	-8.88
			283	55.3	39.1	47.5	13.4	60.34	-5.04
			284	62.8	31.8	51.2	17.0	63.90	-1.10
			285	63.6	31.8	52.0	16.2	63.64	-.04
			286	78.7	14.0	45.8	40.2	76.36	2.34
			287	70.0	23.0	57.2	19.8	67.92	2.08
			288	91.8	11.8	32.5	55.7	81.24	10.56
			289	90.4	23.3	47.6	29.1	70.26	20.14
Sheridan County State Lake.	Kansas.....	USBR.....	290	50.5	33.7	55.7	10.6	61.62	-11.12
			291	49.5	46.9	51.8	1.3	54.34	-4.84
			292	71.0	13.8	64.5	21.7	71.68	-.68
			293	64.5	24.2	64.4	11.4	65.22	-.72
			294	63.4	26.1	67.3	6.6	63.46	-.06
			295	61.3	27.7	67.8	4.5	62.20	-.90
			296	67.7	21.9	68.7	9.4	65.42	2.28
			297	71.0	21.0	55.8	23.2	69.42	1.58
			298	95.7	0	1.3	99.7	97.00	-1.30
			299	89.2	20.3	64.9	14.8	67.70	21.50
			300	82.3	40.8	50.5	8.7	58.58	23.72
			301	81.9	41.9	48.0	10.1	58.48	23.42
Lake Avalon.....	New Mexico.....	USBR.....	302	76.4	33.2	37.5	29.3	66.66	9.74
			303	84.5	52.1	39.7	8.2	54.36	30.14
			304	68.1	35.7	46.4	17.9	62.72	5.38
			305	86.7	22.7	45.7	31.6	71.04	15.66
			306	77.4	35.6	57.1	7.3	59.86	17.54
			307	41.37	35.0	47.0	18.0	63.08	-21.71
			308	84.89	1.0	59.0	40.0	81.04	3.85
			309	57.77	28.0	58.0	14.0	64.56	-6.79
			310	67.33	27.0	68.0	5.0	62.58	4.75
			311	91.53	1.0	87.0	12.0	73.76	17.77
			312	60.42	28.0	59.0	13.0	64.30	-3.88
			313	37.13	21.0	58.0	21.0	68.90	-31.77
John Martin Reservoir.	Colorado.....	CE.....	314	62.54	37.0	55.0	8.0	59.76	2.78
			315	83.97	3.0	56.0	41.0	80.58	3.39
			316	80.89	14.0	64.0	22.0	71.68	9.21
			317	53.15	32.0	63.0	5.0	60.78	-7.63
			318	91.50	11.0	41.0	48.0	79.52	11.98
			319	61.62	39.0	59.0	2.0	57.48	4.14
			320	81.50	19.0	74.0	7.0	65.98	15.52
			321	80.57	11.0	78.0	11.0	69.90	10.67
			322	87.52	3.0	41.0	56.0	84.48	3.04
			323	97.07	0	8.0	92.0	94.92	2.15
			324	79.3	6.0	42.0	52.0	82.36	-3.06
			325	73.6	14.0	68.0	18.0	70.64	2.96
			326	65.8	16.0	63.0	21.0	70.70	-4.90
			327	96.1	4.0	34.0	62.0	85.68	10.42
			328	97.6	5.0	40.0	55.0	83.50	14.10
			329	89.3	10.0	51.0	39.0	77.54	11.76
			330	114.0	5.0	32.0	63.0	85.58	28.42
			331	89.7	6.0	36.0	58.0	83.92	5.78
			332	80.5	11.0	49.0	40.0	77.44	3.06
			333	102.5	1.0	8.0	91.0	94.30	8.20
			334	70.6	15.0	47.0	38.0	75.48	-4.88
			335	69.2	21.0	72.0	7.0	65.26	3.94
			336	97.3	5.0	40.0	55.0	83.50	13.80
			337	105.2	1.0	10.0	89.0	93.78	11.42
			338	101.6	2.0	8.0	90.0	93.68	7.92
			339	79.6	15.0	84.0	1.0	65.86	13.74
			340	102.9	1.0	2.0	97.0	95.86	7.04
			341	47.8	65.0	34.0	1.0	47.86	-.06
			342	41.9	60.0	37.0	3.0	50.18	-8.28
			343	40.1	58.0	39.0	3.0	50.90	-10.80
			344	44.6	60.0	37.0	3.0	50.18	-5.58
			345	38.5	63.0	35.0	2.0	48.84	-10.34
			346	43.9	63.0	36.0	1.0	48.58	-4.68

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
John Martin Reservoir—Con.	Colorado.....	CE.....	347	<i>Lb./cu. ft.</i> 44.9	39.0	55.0	6.0	58.52	-13.62
			348	41.6	58.0	40.0	2.0	50.64	-9.04
			349	43.1	49.0	51.0	0	53.36	-10.26
			350	42.6	45.0	53.0	2.0	55.32	-12.72
McMillan Reservoir..	New Mexico.....	USBR.....	351	41.2	71.0	29.0	0	45.44	-4.24
			352	57.8	71.8	25.1	3.1	45.86	11.94
			353	64.8	73.5	24.7	1.8	45.01	19.79
			354	60.3	77.5	21.1	1.4	43.46	16.84
			355	63.9	73.2	25.8	1.0	44.98	18.92
			356	63.0	75.4	23.6	1.0	44.26	18.74
			357	67.9	79.0	20.9	.1	42.56	25.34
			358	76.9	68.1	29.4	2.5	47.13	29.77
			359	55.8	77.6	22.4	0	42.92	12.88
			360	60.5	72.1	25.8	2.1	45.60	14.90
			361	60.6	70.3	29.7	0	45.80	14.80
			362	62.4	73.9	26.1	0	44.36	18.04
			363	63.1	73.1	25.9	1.0	44.98	18.12
			364	51.2	62.8	37.1	.1	48.32	2.88
			365	57.4	85.5	14.3	.2	40.27	17.13
			366	60.5	72.8	25.7	1.5	45.18	15.32
			367	72.4	79.9	18.0	2.1	42.72	29.68
			368	56.2	78.9	20.6	.5	42.73	13.47
			369	61.8	70.7	28.8	.5	45.68	16.12
			370	63.6	67.1	32.7	.2	46.88	16.72
Settling Basin Socorro Main Canal.	New Mexico.....	USBR.....	371	65.5	54.2	44.4	1.4	51.82	13.68
			372	64.3	63.2	36.2	.6	48.58	15.72
			373	88.6	17.7	74.9	7.4	66.34	22.26
			374	75.5	10.4	38.2	51.4	80.66	-5.16
			375	76.8	9.5	16.6	73.9	86.79	9.99
			376	73.6	25.5	66.2	8.3	63.98	9.62
			377	84.9	12.1	28.9	59.0	82.02	2.88
			378	79.9	17.2	43.2	39.6	75.28	4.62
			379	59.3	4.0	14.2	81.8	90.88	-31.58
			380	81.1	22.7	58.1	19.2	67.66	13.44
			381	68.0	13.4	36.3	50.3	79.32	-11.32
			382	71.8	9.3	21.3	69.4	85.70	-13.90
			383	86.7	18.6	64.3	17.1	68.58	18.12
			384	78.6	14.1	43.5	42.4	76.88	1.72
			385	71.1	7.5	9.5	83.0	89.88	-18.78
			386	87.4	3.9	27.5	68.6	87.50	-.10
			387	81.7	6.8	2.7	90.5	92.08	-10.38
			388	81.1	5.8	13.6	80.6	89.90	-8.80
			389	63.0	9.1	14.4	76.5	87.61	-24.61
			390	96.1	5.3	6.5	88.2	92.08	4.02
			391	68.6	53.0	46.4	0.6	52.18	16.42
			392	76.1	20.2	20.3	59.5	79.20	-3.10
			393	55.5	26.6	69.4	4.0	62.32	-6.82
			394	74.3	12.0	12.4	75.6	86.44	-12.44
			395	67.4	16.4	29.8	53.8	79.28	-11.88
			396	68.6	26.2	58.5	15.3	65.54	3.06
			397	88.6	9.6	56.1	34.3	76.24	12.36
			398	76.8	18.7	57.4	23.9	70.40	6.40
			399	64.9	34.4	47.2	18.4	63.44	1.46
			400	84.2	26.4	47.6	26.0	68.40	15.80
			401	68.0	32.7	47.3	20.0	64.32	3.68
			402	60.5	10.1	13.7	76.2	87.16	-26.66
			403	94.2	5.3	8.3	86.4	91.56	2.64
			404	74.9	7.5	10.9	81.6	89.52	-14.62
			405	69.3	8.9	18.2	72.9	86.74	-17.44
			406	86.1	21.2	55.0	23.8	69.68	16.42
			407	74.9	13.0	28.1	58.9	81.66	-6.76
			408	69.3	8.7	10.6	80.7	88.82	-19.52
			409	79.9	25.0	53.8	21.2	67.46	12.44
			410	74.9	18.0	35.0	47.0	76.74	-1.84
			411	57.4	9.5	15.7	74.8	87.03	-29.63
			412	61.2	9.2	13.0	77.8	88.04	-26.84
			413	85.5	24.2	56.8	19.0	67.30	18.20
			414	65.5	13.4	25.4	61.2	82.18	-16.68

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Lake Sabetha.....	Kansas.....	ARS.....		<i>Lb./cu. ft.</i>				<i>Lb./cu. ft.</i>	
			415	33.4	73.8	26.1	.1	44.36	-10.96
			416	47.8	54.8	44.9	.3	51.20	-3.40
			417	41.8	65.0	34.9	.1	47.60	-5.80
			418	59.9	37.9	61.8	.3	57.32	2.58
			419	63.8	27.6	70.4	2.0	61.44	2.36
			420	50.4	47.5	51.7	.8	54.11	-3.71
			421	65.1	31.3	62.3	6.4	61.40	3.70
			422	74.9	39.5	57.0	3.5	57.69	17.21
			423	76.4	27.6	71.3	1.1	61.18	15.22
			424	73.5	24.4	74.7	.9	62.62	10.88
			425	71.4	23.3	75.8	.9	62.98	8.42
			426	82.7	22.3	76.5	1.2	63.34	19.36
			427	46.7	86.9	9.5	3.6	40.72	5.98
			428	46.1	86.7	10.3	3.0	40.46	5.64
			429	38.1	88.9	8.1	3.0	39.74	-1.64
			430	40.5	86.7	9.5	3.8	40.72	-.22
			431	40.5	87.5	8.2	4.3	40.62	-.12
			432	39.3	85.7	10.4	3.9	41.08	-1.78
			433	41.2	82.5	7.3	10.2	43.95	-2.75
			434	47.4	80.3	12.4	7.3	44.02	3.38
			435	37.4	89.0	7.5	3.5	39.87	-2.47
			436	42.4	83.7	10.2	6.1	42.32	.08
			437	49.3	82.1	10.5	7.4	43.30	6.00
			438	51.1	79.2	13.9	6.9	44.38	6.72
			439	47.4	93.8	3.0	3.2	37.94	9.46
			440	65.5	70.7	18.3	11.0	48.30	17.20
			441	68.6	68.1	18.1	13.8	50.16	18.44
			442	68.6	68.7	19.5	11.8	49.28	19.32
			443	56.8	75.4	19.6	5.0	45.30	11.50
			444	63.0	76.2	16.9	6.9	45.46	17.54
			445	39.9	72.6	18.2	9.2	47.06	-7.16
			446	71.8	71.7	17.6	10.7	47.94	23.86
			447	56.1	69.4	18.8	11.8	49.28	6.82
			448	81.1	66.9	19.6	13.5	50.43	30.67
			449	80.5	62.6	17.5	19.9	53.52	26.98
			450	39.9	74.6	14.9	10.5	46.87	-6.97
			451	49.9	75.2	15.6	9.2	46.34	3.56
			452	44.3	80.6	14.8	4.6	43.14	1.16
			453	48.0	75.1	14.3	10.6	46.86	1.14
			454	53.0	78.8	14.3	6.9	44.38	8.62
			455	48.7	80.3	17.2	2.5	42.74	5.96
			456	68.0	71.2	19.0	9.8	48.04	19.96
			457	78.0	58.0	11.4	30.6	58.18	19.82
			458	75.0	66.8	18.9	14.3	50.52	24.48
			459	40.5	84.3	11.0	4.7	42.06	-1.56
			460	31.2	84.6	9.5	5.9	41.96	-10.76
			461	76.8	67.1	18.1	14.8	50.78	26.02
			462	86.9	60.3	16.8	22.9	55.38	31.52

TYPE III. — RESERVOIR OPERATION: RESERVOIR NORMALLY EMPTY

Lake Claremore.....	Oklahoma.....	SCS.....	1	43.0	76.0	22.9	1.1	47.93	-4.93
			2	45.0	72.4	27.4	.2	48.96	-3.96
			3	40.0	71.8	27.7	.5	49.15	9.15
			4	44.0	61.0	38.8	.2	52.48	-8.48
			5	51.0	41.7	57.9	.4	58.56	-7.56
			6	55.0	45.4	53.4	1.2	57.85	-2.85
			7	54.0	39.9	59.0	1.1	59.45	-5.45
			8	63.0	32.1	65.1	2.8	62.51	.49
			9	51.0	50.2	49.4	.4	56.00	5.00
Grisham Lake.....	Missouri.....	SCS.....	10	65.0	24.6	53.8	21.6	69.50	-4.50
			11	53.0	28.6	67.4	4.0	63.72	-10.72
			12	56.0	25.1	66.8	8.1	66.00	-10.00
High Point Reservoir.	North Carolina....	SCS.....	13	117.0	0	2.2	97.8	96.50	20.50
			14	59.0	24.3	53.6	22.1	69.82	-10.82
			15	62.0	20.4	56.6	23.0	71.35	-9.35
			16	68.0	68.6	31.0	.4	49.92	18.08
			17	41.0	54.0	45.0	1.0	54.97	-13.97

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
High Point Reservoir Continued	North Carolina	SCS	18	<i>Lb./cu. ft.</i> 60.0	31.8	65.6	2.6	62.51	-2.51
			19	44.0	28.6	66.0	5.4	63.97	-19.97
			20	64.0	76.7	23.0	.3	47.36	16.64
			21	45.0	25.6	62.1	12.3	66.68	-21.68
			22	37.0	41.8	56.8	1.4	58.81	-21.81
Kirk Lake	Kansas	SCS	23	40.0	83.1	15.3	1.6	45.94	-5.94
			24	42.0	63.1	35.8	1.1	52.09	-10.09
Lake Lee	North Carolina	SCS	25	55.0	44.6	53.8	1.6	58.10	-3.10
			26	62.0	55.9	41.4	2.7	54.83	7.17
			27	60.0	59.3	39.0	1.7	53.62	6.38
			28	59.0	49.5	49.7	.8	56.36	2.64
			29	59.0	37.3	60.8	1.9	60.66	-1.66
Lake Marinuka	Wisconsin	SCS	30	60.0	42.8	55.3	1.9	58.74	1.26
			31	61.0	40.6	58.0	1.4	59.13	1.87
			32	66.0	40.8	58.1	1.1	59.13	6.87
			33	73.0	28.8	69.2	2.0	63.22	9.78
			34	40.0	63.8	34.8	1.4	51.77	-11.77
			35	52.0	67.7	32.3	0	50.24	1.76
			36	56.0	40.6	59.1	.3	58.88	-2.88
			37	55.0	39.9	59.8	.3	59.20	-4.20
			38	60.0	30.8	68.6	.6	62.33	-2.33
			39	70.0	33.3	63.5	3.2	62.19	-7.81
			40	63.0	36.9	62.8	.5	60.32	2.68
			41	60.0	29.4	69.5	1.1	62.97	-2.97
			42	66.0	23.1	72.3	4.6	65.89	.11
			43	80.0	11.7	70.6	16.7	72.41	7.59
			44	77.0	18.8	73.5	7.7	67.92	9.08
Shepherd Mt. Lake	Missouri	SCS	45	69.0	18.8	64.1	17.1	70.17	-1.17
			46	85.0	9.2	68.4	22.4	74.62	10.38
			47	87.0	16.5	72.3	11.2	69.52	17.48
Moran Reservoir	Kansas	SCS	48	43.0	20.0	64.8	15.2	69.35	-26.35
			49	85.0	0	1.4	99.6	97.00	-12.00
			50	50.0	62.8	36.8	.4	51.84	-1.84
			51	38.0	69.0	30.3	.7	50.17	-12.17
			52	52.0	46.5	52.4	1.1	57.40	-5.40
Mountain Lake	Missouri	SCS	53	62.0	24.6	56.1	19.3	68.75	-6.75
			54	49.0	41.6	57.3	1.1	58.81	-9.81
			55	43.0	65.0	34.7	.3	51.20	-8.20
Neosho County State Lake	Kansas	SCS	56	57.0	33.0	62.2	4.8	62.69	-5.69
			57	66.0	18.7	78.2	3.1	66.67	-0.67
			58	37.0	74.8	23.2	2.0	48.50	-11.50
			59	30.0	64.3	34.1	1.6	52.02	-22.02
			60	36.0	43.9	55.4	.7	58.17	-22.17
Grand Saline	Texas	SCS	61	38.0	47.8	49.9	2.3	57.14	-19.14
			62	45.0	27.7	68.5	3.8	64.04	-19.04
			63	40.0	67.0	32.0	1.0	50.81	-10.81
			64	35.0	68.0	30.0	2.0	50.74	-15.74
			65	47.0	33.0	53.0	14.0	64.94	-17.94
Wills Point Reservoir	Texas	SCS	66	30.0	60.0	34.0	6.0	54.30	-24.30
			67	34.0	54.0	43.0	3.0	55.47	-21.47
			68	34.0	58.0	40.0	2.0	53.94	-19.94
			69	47.0	34.0	59.0	7.0	62.87	-15.87
			70	43.0	43.0	50.0	7.0	59.99	-16.99
			71	51.0	52.0	46.0	2.0	55.86	-4.86
			72	53.0	55.0	42.0	3.0	55.15	-2.15
			73	53.0	50.0	48.0	2.0	56.50	-3.50
			74	59.0	54.0	45.0	1.0	54.97	4.03
			75	54.0	60.0	38.0	2.0	53.30	0.70
Miami Cons. Dist. Germantown Res. Lake Arthur	South Africa	SCS	76	86.0	26.0	57.0	17.0	67.93	18.07
			77	85.0	26.0	52.0	22.0	69.18	15.82
			78	45.0	73.0	26.0	1.0	48.89	-3.89
			79	52.0	71.0	27.0	2.0	49.78	2.22
			80	80.0	29.0	71.0	0	62.72	17.28
			81	79.0	30.0	68.0	2.0	62.90	16.10
			82	38.0	73.0	27.0	0	48.64	-10.64
			83	39.0	61.0	39.0	0	52.48	-13.48
			84	58.0	61.0	39.0	0	52.48	5.52
			85	100.0	3.0	13.0	84.0	92.04	7.96

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Lake Arthur— Continued	South Africa		86	<i>Lb./cu. ft.</i> 72.0	36.0	35.0	29.0	<i>Lb./cu. ft.</i> 67.73	4.27
			87	64.0	46.0	51.0	3.0	58.03	5.97
			88	100.0	3.0	19.0	78.0	90.54	9.46
			89	80.0	20.0	51.0	29.0	72.85	7.15
Colorado River above Laguna Dam	Arizona	USBR	90	100.0	2.0	72.0	26.0	77.86	22.14
			91	86.0	9.0	31.0	60.0	84.12	1.88
			92	83.0	2.0	14.0	84.0	92.36	-9.36
			93	88.0	3.0	13.0	84.0	92.04	-4.04
			94	81.0	9.0	60.0	31.0	76.87	4.13
			95	86.0	8.0	50.0	42.0	79.94	6.06
			96	72.0	24.0	60.0	26.0	70.82	1.18
			97	87.0	0	10.0	90.0	94.50	-7.50
			98	64.0	30.0	55.0	15.0	66.15	-2.15
			99	61.2	23.0	15.0	62.0	80.14	-18.94
Sirianni Lake	Texas	SCS	100	30.0	69.0	26.0	5.0	51.17	-21.17
Smith Pond	Texas	SCS	101	28.7	69.0	29.0	2.0	50.42	-21.72
XGZ Ranch Pond	Texas	SCS	102	26.8	71.0	27.0	2.0	49.78	-22.98
Harold Fress Pond	Texas	SCS	103	55.5	63.0	36.0	1.0	52.09	3.41
			104	71.8	51.0	46.0	3.0	56.43	15.37
Lake Eddleman	Texas	SCS	105	84.9	41.0	47.0	12.0	61.88	23.02
			106	54.3	37.0	51.0	12.0	63.16	-8.86
White Rock Lake	Texas	SCS	107	34.5	36.0	14.0	0	60.48	-25.98
			108	36.4	60.0	39.0	1.0	53.05	-16.65
			109	52.7	41.0	55.0	4.0	59.88	-7.18
Lake Crook	Texas	SCS	110	30.6	91.6	9.0	0	42.88	-12.28
			111	32.4	92.0	8.0	0	42.56	-10.16
			112	31.8	91.0	9.0	0	42.88	-11.08
			113	40.5	91.0	9.0	0	42.88	-2.38
			114	41.3	85.0	15.0	0	44.80	-3.50
			115	30.7	83.0	16.0	1	45.69	-14.99
			116	43.0	85.0	15.0	0	44.80	-1.80
Dawson City Lake	Texas	SCS	117	27.5	89.0	11.0	0	43.52	-16.02
			118	44.5	67.0	33.0	0	50.56	-6.06
			119	29.7	82.0	18.0	0	45.76	-16.06
Kerens City Lake	Texas	SCS	120	32.2	78.0	22.0	0	47.04	-14.84
			121	36.7	69.0	30.0	1.0	50.17	-13.47
			122	76.1	19.0	26.0	55.0	79.67	-3.57
			123	59.2	51.0	46.0	3.0	56.43	2.77
Loring Lake	Louisiana	SCS	124	33.8	70.0	28.0	2.0	50.10	-16.30
			125	48.9	52.0	38.0	10.0	57.86	-8.96
			126	81.5	15.0	31.0	54.0	80.70	.80
			127	89.6	12.0	37.0	51.0	80.91	8.69
			128	74.0	8.0	11.0	81.0	89.69	-15.69
Jenkins Pond	Louisiana	SCS	129	92.2	6.0	20.0	74.0	88.58	3.62
			130	88.0	8.0	24.0	68.0	86.44	1.56
			131	69.4	14.0	23.0	63.0	83.27	-13.87
Sandstone Creek Structure No. 5. Structure No. 6.	Oklahoma	SCS	132	88.0	9.0	19.0	72.0	87.12	.88
			133	79.5	8.0	15.0	77.0	88.69	-9.19
			134	83.0	24.0	37.0	39.0	74.07	8.93
			135	68.6	37.0	40.0	23.0	65.91	2.69
			136	92.0	18.0	23.0	59.0	80.99	11.01
			137	86.7	17.0	17.0	66.0	83.06	3.64
			138	99.3	1.0	0.0	99.0	96.43	2.87
Sandstone Creek Structure No. 9.		SCS	139	77.2	19.0	39.0	42.0	76.42	.78
			140	67.4	41.0	48.0	11.0	61.63	5.77
			141	70.3	42.0	52.0	6.0	60.06	10.24
			142	87.9	15.0	29.0	56.0	81.20	6.70
			143	79.4	12.0	17.0	71.0	85.91	-6.51
Sandstone Creek Structure No. 16.			144	73.9	17.0	30.0	53.0	79.81	-5.91
			145	68.5	34.0	47.0	19.0	65.87	2.63
			146	99.5	15.0	39.0	46.0	78.70	20.80
			147	62.9	25.0	42.0	33.0	72.25	-9.25
			148	70.4	26.0	44.0	30.0	71.18	-.78
Sandstone Creek Structure No. 22.			149	83.2	17.0	45.0	38.0	76.06	7.14
			150	83.2	15.0	25.0	60.0	82.20	1.00
			151	74.4	18.0	47.0	35.0	74.99	-.59
			152	80.7	26.0	61.0	13.0	66.93	13.77
			153	78.8	24.0	53.0	23.0	70.07	8.73

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Sandstone Creek Structure No. 1.			154	<i>Lb./cu. ft.</i> 45.7	42.0	49.0	9.0	60.81	-15.11
			155	54.9	30.0	56.0	14.0	65.90	-11.00
			156	75.2	34.0	60.0	6.0	62.62	12.58
			157	80.8	22.0	58.0	20.0	69.96	10.84
Sandstone Creek Structure No. 14			158	61.7	26.0	54.0	20.0	68.68	-6.98
			159	65.6	24.0	47.0	29.0	71.57	-5.79
			160	76.5	17.0	43.0	40.0	76.56	-.06
			161	78.2	17.0	53.0	30.0	74.06	4.14
Sandstone Creek Structure No. 17			162	90.8	9.0	8.0	83.0	89.87	.93
			163	83.2	9.0	35.0	56.0	83.12	.08
			164	84.6	27.0	43.0	30.0	70.86	13.74
			165	60.8	30.0	51.0	19.0	67.15	-6.35
Lake Fort, Phantom Hill.	Texas.....	SCS.....	166	58.7	46.0	38.0	16.0	61.28	-2.58
			167	29.3	82.0	17.0	1.0	46.01	-16.71
			168	31.6	74.0	25.0	1.0	48.57	-16.97
			169	29.9	76.0	23.0	1.0	47.93	-18.03
			170	42.4	42.0	52.0	6.0	60.06	-17.66
			171	28.4	82.0	71.0	1.0	46.01	-17.61
			172	32.6	80.0	20.0	0	46.40	-13.80
			173	44.3	44.0	45.0	11.0	60.67	-16.37
			174	50.5	39.0	50.0	11.0	62.27	-11.77
			175	41.4	41.0	53.0	6.0	60.38	-18.98
			176	29.0	73.0	25.0	2.0	49.14	-20.14
			177	36.4	49.0	48.0	3.0	57.07	-20.67
Lake Throckmorton..	Texas.....	SCS.....	178	26.7	59.0	39.0	2.0	53.62	-26.92
			179	33.0	60.0	39.0	1.0	53.05	-20.05
			180	32.9	59.0	40.0	1.0	53.37	-20.47
			181	31.3	62.0	37.0	1.0	52.41	-21.11
George Johnson Pond No. P-2.	South Dakota.....	SCS.....	182	33.8	56.0	42.0	2.0	54.58	-20.78
			183	79.3	53.0	25.0	22.0	60.54	18.76
			184	72.5	54.0	28.0	18.0	59.22	13.28
			185	86.1	37.0	29.0	34.0	63.66	17.44
Bertha Armour Pond No. P-8.	South Dakota.....	SCS.....	186	82.5	45.0	28.0	27.0	64.35	18.15
			187	84.3	43.0	26.0	31.0	65.99	18.31
			188	81.1	45.0	23.0	32.0	65.60	15.50
			189	78.6	31.0	29.0	40.0	72.08	6.62
Charles Crago Charles Crago Pond No. P-10.	South Dakota.....	SCS.....	190	93.6	27.0	28.0	45.0	74.61	18.99
			191	91.9	16.0	29.0	55.0	80.63	11.27
			192	41.8	77.0	16.0	7.0	49.11	-7.31
			192	41.8	77.0	16.0	7.0	49.11	-7.31
Charles Crago Pond No. P-11.	South Dakota.....	SCS.....	193	29.3	84.0	13.0	3.0	45.87	-16.57
			194	29.9	83.0	13.0	4.0	46.44	-16.54
			195	71.1	66.0	20.0	14.0	54.38	16.72
			196	58.0	81.0	14.0	5.0	47.33	10.67
M. I. Hefner Pond No. P-12. Ashland Reservoir....	South Dakota.....	SCS.....	197	39.9	85.0	9.0	6.0	46.30	-6.40
			198	47.4	79.0	18.0	3.0	47.47	-.07
			199	64.9	87.0	8.0	5.0	45.41	19.49
			200	36.2	88.0	11.0	1.0	44.09	-7.89
			201	53.6	90.0	9.0	1.0	43.45	10.15
			202	43.7	85.0	12.0	3.0	45.55	-1.85
			203	48.0	82.0	15.0	3.0	46.51	1.49
			204	67.5	77.0	18.0	5.0	48.61	18.89
			205	80.0	69.0	15.0	16.0	53.92	26.08
			206	84.0	4.0	51.0	45.0	81.97	2.03
			207	80.0	2.0	48.0	50.0	83.86	-3.86
			208	82.0	4.0	78.0	18.0	75.22	6.78
	Missouri.....	SCS.....	209	67.0	5.0	82.0	13.0	73.65	-6.65
			210	62.0	5.0	86.0	9.0	72.65	-10.65
			211	61.0	8.0	83.0	9.0	71.69	-10.69
			212	54.0	26.0	64.0	10.0	66.18	-12.18
			213	85.0	5.0	69.0	26.0	76.90	8.10
			214	80.0	8.0	77.0	15.0	73.19	6.81
			215	78.0	8.0	79.0	13.0	72.69	5.31
			216	69.0	11.0	75.0	14.0	71.98	-2.98
			217	62.0	12.0	75.0	13.0	71.41	-9.41
			218	57.0	19.0	63.0	18.0	70.42	-13.42
			219	55.0	29.0	61.0	10.0	65.22	-10.22
			220	57.0	28.0	61.0	11.0	65.79	-8.79

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Pond No. 1.....	Illinois.....	Illinois State Water Survey.	221	<i>Lb./cu. ft.</i> 51.79	75.7	42.2	.1	47.68	4.11
			222	65.52	63.7	21.9	14.4	55.02	10.20
			223	66.77	56.9	21.1	22.0	59.26	7.51
			224	39.31	80.8	18.3	.9	46.33	-7.02
			225	60.53	77.0	21.7	1.3	47.61	12.92
			226	59.28	80.0	17.1	2.9	47.15	12.13
			227	69.26	65.2	27.1	7.7	53.20	16.06
			228	62.40	71.6	23.4	5.0	50.21	12.19
			229	67.39	64.7	24.8	10.5	53.92	13.47
			230	73.63	65.2	21.4	13.4	54.45	19.18
			231	53.66	76.7	21.7	1.6	47.86	5.80
			232	74.26	64.0	20.6	15.4	55.27	18.99
			233	48.67	79.4	19.2	1.4	46.97	1.70
			234	32.45	85.0	14.3	.7	45.05	-12.60
Pond No. 4.....	Illinois.....	Illinois State Water Survey.	235	68.64	84.5	14.5	1.0	45.21	23.43
			236	40.56	78.7	18.2	3.1	47.47	-6.91
Pond No. 5.....	Illinois.....	Illinois State Water Survey.	237	76.13	52.0	15.5	32.5	63.48	12.65
			238	59.28	65.7	22.4	11.9	53.88	5.40
			239	67.39	74.3	21.2	4.5	49.35	18.04
			240	55.54	78.1	21.2	.7	47.29	8.25
			241	39.31	79.3	20.3	.4	46.72	-7.41
			242	62.40	71.8	27.5	.7	49.21	13.19
			243	42.43	82.0	17.5	.5	45.88	-3.45
			244	38.06	80.4	18.7	.9	46.65	-8.59
			245	53.66	79.8	19.3	.9	46.65	7.01
			246	68.02	75.1	22.8	2.2	48.50	19.52
			247	56.78	70.1	28.2	1.7	50.10	6.68
			248	54.28	74.1	22.8	3.1	49.07	5.21
			249	70.51	65.9	22.3	11.8	53.88	16.63
			250	66.70	70.4	23.9	5.7	51.10	15.60
			251	48.67	10.7	13.6	5.7	47.58	1.09
			252	48.67	79.8	15.2	5.0	47.65	1.02
			253	59.90	75.1	24.3	.6	48.25	11.65
			254	69.89	76.4	21.4	2.2	48.18	21.71
			255	64.27	64.8	24.9	10.3	53.70	10.57
			256	58.65	81.8	17.5	.7	46.01	12.64
			257	50.54	70.5	23.6	5.9	50.92	-.38
			258	66.14	67.3	24.1	8.6	52.81	13.33
			259	66.77	64.5	32.7	2.8	52.06	14.71
			260	42.38	62.9	29.1	8.0	53.84	-11.46
			261	46.80	81.9	17.9	.2	45.76	1.04
			262	61.77	74.5	22.2	3.3	48.98	12.79
			263	52.41	74.2	24.5	1.3	48.57	3.84
			264	54.29	75.0	24.6	.4	48.00	6.29
			265	63.02	76.9	22.6	.5	47.52	15.50
			266	61.77	74.8	22.7	.5	48.19	13.58
			267	29.33	78.0	18.3	3.7	48.04	-18.71
			268	29.33	73.8	25.8	.4	48.32	-18.99
			269	44.93	80.4	17.9	1.7	46.90	-1.97
			270	58.03	71.8	24.4	3.8	49.96	8.07
			271	58.03	76.4	22.3	1.3	47.93	10.10
			272	28.08	80.9	17.8	1.3	46.33	-18.25
			273	45.55	78.9	19.9	1.2	46.97	-1.42
			274	49.30	74.9	22.3	2.8	48.75	.55
			275	30.58	85.6	14.3	.1	44.48	-13.90
			276	48.67	50.2	12.5	37.3	65.25	-16.58
			277	43.06	79.1	19.0	1.9	47.22	-4.16
			278	30.58	78.0	16.4	5.6	48.54	-17.96
			279	48.67	86.2	13.0	.8	44.73	3.94
			280	44.30	84.9	14.6	.5	44.96	-.66
			281	41.81	76.0	23.5	.5	47.80	-5.99
			282	45.55	87.5	12.0	.5	44.12	1.43
			283	57.41	73.7	23.3	3.0	49.07	8.34
			284	52.42	67.7	29.9	2.4	50.74	1.63
			285	68.64	64.4	22.9	12.7	54.77	13.87
			286	79.25	62.4	22.8	14.8	55.91	23.34
			287	52.42	69.4	28.0	2.6	50.67	1.75

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Pond No. 20.....	Illinois.....	Illinois State Water Survey.	288	<i>Lb./cu. ft.</i> 34.32	86.1	12.9	1.0	<i>Lb./cu. ft.</i> 44.73	-10.41
			289	31.82	82.7	16.2	1.1	45.69	-13.87
			290	49.92	80.1	18.7	1.2	46.65	3.27
			291	46.80	74.1	19.7	6.2	49.82	-3.02
			292	36.82	78.9	20.6	.5	46.88	-10.06
			293	46.18	75.6	22.1	2.3	48.18	-2.00
			294	51.17	78.1	21.3	.6	47.29	3.88
			295	77.38	59.3	33.8	6.9	54.87	22.51
			296	63.65	68.0	29.3	2.7	50.99	12.66
Pond No. 21.....	Illinois.....	Illinois State Water Survey.	297	63.65	72.8	25.8	1.4	48.89	14.76
			298	38.69	87.0	12.0	1.0	44.41	-5.72
			299	38.69	83.3	16.2	.5	45.47	-6.78
			300	37.44	84.7	14.6	.7	45.05	-7.61
			301	41.18	84.6	14.9	.5	45.05	-3.87
			302	47.42	83.9	15.8	.3	45.1	2.30
			303	62.40	76.2	23.4	.4	47.68	14.72
			304	66.77	74.2	24.6	1.2	48.57	18.20
			305	72.38	67.1	26.7	6.2	52.00	20.32
			306	67.39	73.6	24.3	2.1	48.82	18.57
Pond No. 24.....	Illinois.....	Illinois State Water Survey.	307	74.26	65.9	27.6	6.5	52.54	21.72
			308	40.56	66.2	17.2	16.6	55.13	-14.57
			309	31.82	76.3	21.4	2.3	48.18	-16.36
			310	33.69	78.3	18.9	2.8	47.79	-14.10
			311	36.19	73.3	22.3	4.4	49.64	-13.45
			312	34.32	77.0	20.3	2.7	48.11	-13.79
			313	35.57	79.4	18.6	2.0	47.22	-11.65
			314	38.06	72.8	23.0	4.2	49.54	-11.58
			315	46.18	74.7	19.7	5.6	49.50	-3.32
Pond No. 25.....	Illinois.....	Illinois State Water Survey.	316	43.68	78.6	19.6	1.8	47.22	-3.54
			317	41.18	72.3	23.1	4.6	50.21	-9.03
			318	66.14	63.8	24.0	12.2	54.52	11.62
			319	70.51	60.6	21.6	17.8	56.98	13.53
			320	39.31	79.4	19.1	1.5	46.97	-7.66
Pond No. 29.....	Illinois.....	Illinois State Water Survey.	321	52.42	75.6	22.1	2.3	48.18	4.24
			322	36.19	76.8	22.3	.9	47.61	-11.42
			323	39.94	76.0	21.5	2.5	48.30	-8.36
			324	55.66	73.7	23.0	3.3	49.07	6.59
			325	39.31	79.0	19.5	1.5	47.10	-7.79
			326	48.67	78.5	20.3	1.2	47.18	1.49
			327	76.13	65.6	17.8	16.6	55.13	21.00
Pond No. 33.....	Illinois.....	Illinois State Water Survey.	328	73.63	72.7	23.9	3.4	49.39	24.24
			329	61.15	81.2	18.5	.4	46.08	15.07
Pond No. 42.....	Illinois.....	Illinois State Water Survey.	330	65.52	80.1	19.2	.7	46.65	18.87
			331	19.97	77.2	21.7	1.1	47.61	-27.64
			332	38.69	73.8	24.1	2.1	48.82	-10.13
			333	29.95	70.2	26.3	3.5	50.41	-20.46
			334	33.70	72.5	21.3	6.2	50.35	-16.65
			335	48.05	70.8	23.0	6.2	50.78	-2.73
			336	50.54	70.4	26.5	3.1	50.35	.19
			337	49.30	72.9	25.3	1.8	49.14	.16
			338	37.44	69.2	29.7	1.1	50.17	-12.73
			339	53.66	72.1	25.0	2.9	49.71	3.95
Pond No. 44.....	Illinois.....	Illinois State Water Survey.	340	44.30	64.8	31.1	4.1	52.20	-7.90
			341	41.81	75.6	23.6	.8	47.93	-6.12
			342	51.79	79.9	19.2	.9	46.65	5.14
			343	60.53	69.6	27.9	2.5	50.35	10.18
			344	63.65	68.8	27.3	3.9	50.92	12.73
			345	65.52	76.2	22.0	1.8	48.18	17.34
			346	58.66	59.9	35.3	4.8	54.05	4.61
Pond No. 46.....	Illinois.....	Illinois State Water Survey.	347	78.00	66.5	29.4	4.1	51.74	26.26
			348	33.70	86.0	13.0	1.0	44.73	-11.03
			349	40.56	84.7	13.6	1.7	45.30	-4.74
			350	39.94	82.4	16.4	1.2	46.01	-6.07
			351	44.93	84.2	14.3	1.5	45.43	-.50
			352	53.04	79.5	19.7	.8	46.76	6.28
			353	58.66	77.8	20.7	1.5	47.48	11.18

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Thompson Lake.....	Illinois.....	Illinois State Water Survey.	354	<i>Lb./cu. ft.</i> 60.53	76.3	17.1	6.6	<i>Lb./cu. ft.</i> 49.43	11.10
			355	25.58	70.0	25.2	4.8	50.85	25.27
			356	39.31	71.7	22.9	5.4	50.21	10.90
Lake Pana.....	Illinois.....	Illinois State Water Survey.	357	44.93	88.4	11.0	.6	44.09	.84
			358	45.55	90.1	9.3	.6	43.45	2.10
			359	49.92	85.6	13.8	.6	44.73	5.19
			360	38.69	81.0	15.8	3.2	46.83	-8.14
			361	41.81	78.3	20.2	1.5	47.32	-5.51
			362	74.26	76.9	22.7	.4	47.36	26.90
Kahola Reservoir....	Kansas.....	SCS.....	363	37.9	37.0	51.0	12.0	63.16	-25.26
			364	39.8	36.0	56.0	8.0	62.48	-22.68
			365	39.7	34.0	54.0	12.0	64.12	-24.42
			366	42.9	34.0	58.0	8.0	63.12	-20.22
			367	69.7	21.0	73.0	6.0	66.78	2.92
			368	77.5	14.0	80.0	6.0	69.02	8.48
			369	77.0	18.0	79.0	3.0	66.99	10.01
			370	61.3	25.0	70.0	5.0	65.25	-3.95
			371	52.8	23.0	71.0	6.0	66.14	-13.34
			372	68.1	21.0	68.0	11.0	68.03	.07
			373	70.5	18.0	76.0	6.0	67.74	2.76
			374	36.1	40.0	54.0	6.0	60.70	-24.60
			375	54.8	21.0	66.0	13.0	68.53	-13.73
			376	42.8	36.0	47.0	7.0	62.23	-19.43
Whitehead Reservoir..	Nebraska.....	SCS.....	377	50.2	16.0	75.0	9.0	69.13	-18.93
			378	60.5	41.0	48.0	11.0	61.63	-1.13
			379	64.4	41.0	50.0	9.0	61.13	3.27
			380	88.6	24.0	33.0	43.0	75.07	13.53
			381	65.5	35.0	44.0	21.0	66.05	-0.55
			382	82.4	34.0	44.0	22.0	66.62	-15.78
			383	71.2	34.0	46.0	20.0	66.12	5.08
			384	78.0	30.0	40.0	30.0	69.90	8.10
			385	55.5	34.0	53.0	13.0	64.37	-8.87
			386	76.8	36.0	44.0	20.0	65.48	-11.32
			387	51.8	39.0	46.0	15.0	63.27	-11.47
			388	49.9	49.0	43.0	8.0	58.32	-8.42
			389	65.0	28.0	57.0	13.0	66.29	-1.29
			390	59.9	31.0	55.0	14.0	65.58	-5.68
			391	61.8	34.0	46.0	20.0	66.12	-4.32
			392	56.8	40.0	49.0	11.0	61.95	-5.15
			393	47.4	32.0	53.0	15.0	65.51	-18.11
			394	69.4	36.0	45.0	19.0	65.23	4.17
Bruce Dempsey Pond..	Nebraska.....	SCS.....	395	74.3	31.0	48.0	21.0	67.33	6.97
			396	73.0	42.0	40.0	18.0	63.06	9.94
			397	71.8	27.0	45.0	29.0	70.61	1.19
			398	48.0	38.0	46.0	16.0	63.84	-15.84
			399	57.4	30.0	53.0	17.0	66.65	-9.25
			400	83.0	32.0	48.0	20.0	66.76	16.24
			401	75.0	37.0	47.0	16.0	64.16	10.84
			402	80.0	29.0	58.0	13.0	65.97	14.03
			403	75.0	31.0	56.0	13.0	65.33	9.67
			404	80.0	34.0	53.0	13.0	64.37	15.63
			405	79.4	31.0	56.0	13.0	65.33	14.07

TYPE IV. — RIVERBED SEDIMENTS

Missouri River near Kansas City.	Missouri.....		1	81.0	1.0	29.0	70.0	89.67	-8.67
			2	93.0	0	32.0	68.0	89.32	3.68
			3	97.0	0	5.0	95.0	95.80	1.20
			4	93.0	0	34.0	66.0	88.84	4.16
			5	94.0	1.0	13.0	86.0	93.51	.49
			6	88.0	0	38.0	62.0	87.88	.12
			7	72.0	0	45.0	55.0	86.20	-14.20
			8	86.0	1.0	29.0	70.0	89.67	-3.67
			9	84.0	6.0	77.0	17.0	76.30	7.70
			10	62.0	3.0	86.0	11.0	75.25	-13.25
			11	75.0	1.0	98.0	1.0	73.11	1.89
			12	74.0	4.0	95.0	1.0	72.72	1.28
			13	85.0	2.0	76.0	22.0	78.02	6.98

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt. <i>Lb./cu. ft.</i>	Percent			Comp. unit wt. <i>Lb./cu. ft.</i>	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Missouri River near Kansas City—Con.	Missouri		14	72.0	3.0	95.0	2.0	73.09	-1.09
			15	87.0	1.0	93.0	6.0	74.31	12.69
			16	72.0	1.0	89.0	10.0	74.27	-2.27
			17	74.0	6.0	88.0	6.0	73.66	.34
			18	98.0	0	25.0	75.0	91.00	7.00
			19	97.0	0	23.0	77.0	91.48	5.52
			20	85.0	0	23.0	77.0	91.48	-6.48
			21	86.0	0	58.0	42.0	83.08	2.92
			22	82.0	1.0	87.0	12.0	75.75	6.25
			23	80.0	0	86.0	14.0	76.36	3.64
			24	88.0	0	79.0	21.0	78.04	9.96
			25	83.0	0	74.0	26.0	79.24	3.76
			26	83.0	0	57.0	43.0	83.32	-.32
			27	84.0	0	65.0	35.0	81.40	2.60
			28	86.0	0	62.0	38.0	82.12	3.88
			29	81.0	0	71.0	29.0	79.96	1.04
			30	85.0	0	50.0	50.0	85.00	0
			31	96.0	0	18.0	82.0	92.68	3.32
			32	88.0	0	10.0	90.0	94.60	-6.60
			33	89.0	0	9.0	91.0	94.84	-5.84
			34	79.0	0	7.0	93.0	95.32	-16.32
			35	86.0	0	8.0	92.0	95.08	-9.08
			36	90.0	0	8.0	92.0	95.08	-5.08
			37	82.0	0	18.0	82.0	92.68	-10.68
			38	86.0	0	18.0	82.0	92.68	-6.68
			39	82.0	0	14.0	86.0	93.64	-11.64
			40	86.0	0	14.0	86.0	93.64	-7.64
			41	52.0	3.0	50.0	47.0	83.89	-31.89
			42	48.0	2.0	45.0	53.0	85.46	-37.46
			43	43.0	10.	39.0	60.0	87.27	-44.27
			44	45.0	2.0	45.0	53.0	85.46	-40.46
			45	66.0	2.0	36.0	62.0	87.62	-21.62
			46	47.0	3.0	37.0	60.0	87.01	-40.01
			47	57.0	1.0	30.0	69.0	89.43	-32.43
			48	63.0	2.0	35.0	63.0	87.86	-24.86
			49	60.0	2.0	34.0	64.0	88.10	-28.10
Little Colorado River near Grand Falls.	Arizona		50	66.0	1.0	49.0	50.0	84.87	-18.87
			51	91.0	0	6.0	94.0	95.56	-4.56
			52	88.0	3.0	55.0	42.0	82.69	5.31
			53	99.0	1.0	24.0	75.0	90.87	8.13
			54	95.0	3.0	17.0	80.0	91.81	3.19
			55	106.0	1.0	9.0	90.0	94.47	11.53
			56	90.0	2.0	12.0	86.0	93.38	-3.38
			57	94.0	2.0	10.0	88.0	93.86	.14
Middle Rio Grande			58	104.0	2.0	9.0	89.0	94.10	9.90
			59	101.0	.3	1.9	97.8	96.43	4.57
			60	86.1	6.0	92.8	11.2	74.86	11.24
			61	86.9	5.4	61.5	33.1	80.27	6.63
			62	73.2	13.9	84.4	1.7	71.66	1.54
			63	103.5	2.6	26.2	71.2	89.65	13.85
			64	81.7	10.3	85.1	4.1	72.53	9.17
			65	76.4	12.0	86.8	1.2	71.68	4.72
			66	90.3	7.1	33.3	59.6	86.49	3.81
			67	79.6	12.8	84.9	2.3	71.79	7.81
			68	102.0	9.1	8.1	82.8	91.75	10.25
			69	87.0	2.9	32.1	65.0	88.21	-1.21
			70	73.0	38.1	61.4	.5	68.17	4.83
			71	83.1	2.4	7.6	90.0	94.34	-11.24
			72	91.4	8.2	72.0	19.8	76.76	14.64
			73	80.2	7.1	62.6	30.3	79.29	.91
			74	98.0	1.5	1.2	97.3	96.16	1.84
			75	87.0	26.6	46.0	27.4	75.97	11.03
			76	48.3	80.3	19.5	.2	62.60	-14.30
			77	94.2	82.9	16.9	.2	62.21	31.99
			78	81.4	14.0	80.7	5.3	72.38	9.02
			79	101.2	4.6	39.6	55.8	85.79	15.41
			80	88.5	15.3	83.7	1.0	71.29	17.21
			81	80.8	34.6	65.1	.3	68.45	12.35

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt.	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Middle Rio Grande— Continued			82	<i>Lb./cu. ft.</i> 78.6	36.1	63.7	.2	68.32	10.28
			83	78.5	12.6	85.3	2.1	72.03	6.47
			84	80.9	5.4	58.8	35.8	80.99	-.09
Iowa River near Liberty.	Iowa.....		85	69.0	24.0	72.0	4.0	70.84	-1.84
Iowa River near Iowa City.	Iowa.....		86	61.0	16.0	76.0	8.0	72.84	-11.84
			87	56.0	26.0	71.0	3.0	70.34	-14.34
			88	55.0	30.0	67.0	3.0	69.82	-14.82
Iowa River at Iowa City.	Iowa.....		89	56.0	18.0	77.0	5.0	71.86	-15.86
			90	65.0	12.0	61.0	27.0	77.92	-12.92
			91	64.0	14.0	57.0	29.0	78.14	-14.14
			92	52.0	29.0	67.0	4.0	70.19	-18.19
			93	52.0	10.0	89.0	1.0	71.94	-19.94
			94	67.0	18.0	77.0	5.0	71.86	-4.86
			95	64.0	16.0	78.0	6.0	72.36	-8.36
Cedar River near Cedar Valley.	Iowa.....		96	90.0	6.0	28.0	66.0	88.06	1.94
			97	109.0	0	0	100.0	97.00	12.00
			98	99.0	0	0	100.0	97.00	2.00
Colorado River at Yuma.	Arizona.....		99	86.0	2.0	16.0	82.0	92.42	-6.42
			100	88.0	4.0	66.0	30.0	79.68	8.32
			101	84.0	7.0	63.0	30.0	79.29	4.71
			102	93.0	3.0	35.0	62.0	87.49	5.51
			103	69.0	17.0	71.0	12.0	73.67	-4.67
Colorado River at Yuma, Main Canal.	Arizona.....		104	52.0	29.0	50.0	21.0	74.27	-22.27
			105	87.0	1.0	17.0	82.0	92.55	-5.55
			106	85.0	5.0	55.0	40.0	81.95	3.05
Colorado River below Laguna Dam.	Arizona.....		107	85.0	5.0	57.0	38.0	81.47	3.53
			108	88.0	4.0	14.0	82.0	92.16	-4.16
			109	87.0	5.0	13.0	82.0	92.03	-5.03
			110	92.0	2.0	14.0	84.0	92.90	-.90
			111	73.0	6.0	25.0	69.0	88.78	-15.78
			112	97.0	2.0	15.0	83.0	92.66	4.34
			113	82.0	19.0	50.0	31.0	77.97	4.03
			114	76.0	20.0	75.0	5.0	71.60	4.40
			115	82.0	12.0	45.0	43.0	81.76	.24
			116	82.0	12.0	38.0	50.0	83.44	-1.44
			117	93.0	1.0	11.0	88.0	93.99	-.99
			118	93.0	1.0	11.0	88.0	93.99	.99
Colorado River at Yuma.	Arizona.....		119	105.0	0	3.0	97.0	96.28	8.72
			120	98.0	0	3.0	97.0	96.28	1.72
			121	93.0	0	12.0	88.0	94.12	-1.12
			122	89.0	2.0	40.0	48.0	84.26	4.74
			123	95.0	3.0	20.0	77.0	91.09	3.91
			124	78.0	9.0	47.0	44.0	82.39	-4.39
			125	87.0	5.0	30.0	65.0	87.95	-.95
			126	91.0	3.0	20.0	77.0	91.09	-.09
Colorado River above Needles.	California.....		127	92.0	1.0	48.0	51.0	85.11	6.89
			128	96.0	0	12.0	88.0	94.12	1.88
			129	94.0	0	9.0	91.0	94.84	-.84
			130	95.0	0	6.0	94.0	95.56	-.56
			131	94.0	0	5.0	95.0	95.80	-1.80
			132	93.0	0	3.0	97.0	96.28	-3.28
			133	94.0	0	3.0	97.0	96.28	-2.28
			134	67.0	35.0	62.0	3.0	69.17	-2.17
			135	95.0	1.0	49.0	50.0	84.87	10.13
			136	98.0	0	40.0	60.0	87.40	10.60
			137	90.0	3.0	19.0	78.0	91.33	-1.33
			138	86.0	1.0	60.0	39.0	82.23	3.77
Colorado River near Topock.	Arizona.....		139	85.0	0	10.0	90.0	94.60	-9.60
			140	83.0	2.0	42.0	56.0	86.18	-3.18
			141	98.0	0	21.0	79.0	91.96	6.04
			142	88.0	1.0	63.0	36.0	81.51	6.49
			143	94.0	1.0	24.0	75.0	90.87	3.13
			144	98.0	1.0	35.0	65.0	88.47	9.53
			145	73.0	2.0	87.0	11.0	75.38	-2.38
			146	82.0	1.0	65.0	34.0	81.03	.97
			147	70.0	4.0	93.0	3.0	73.20	-3.20
			148	98.0	1.0	37.0	72.0	90.15	7.85
			149	88.0	1.0	50.0	49.0	84.63	3.37

TABLE 1.—*Compilation of data for unit weight analysis—Continued*

Reservoir or lake	Location	Surveyed by	Sample No.	Observed unit wt. <i>Lb./cu. ft.</i>	Percent			Comp. unit wt.	Deviation computed from observed unit weight
					Clay	Silt	Sand		
Colorado River at Yuma—Continued	Arizona		150	84.0	3.0	57.0	40.0	82.21	1.79
			151	87.0	7.0	65.0	28.0	78.81	8.19
			152	88.0	2.0	54.0	44.0	83.30	4.70
			153	87.0	0	70.0	30.0	80.20	6.80
			154	83.0	12.0	57.0	31.0	78.88	4.12
			155	80.0	2.0	38.0	60.0	87.14	-7.14
			156	87.0	1.0	39.0	60.0	87.27	-.27
			157	83.0	1.0	36.0	63.0	87.99	-4.99
			158	87.0	0	34.0	66.0	88.84	-1.84
			159	87.0	0	33.0	67.0	89.08	-2.08
			160	88.0	1.0	34.0	65.0	88.47	-.47
			161	90.0	0	7.0	93.0	95.32	-5.32
			162	92.0	0	5.0	95.0	95.80	-3.80
			163	83.0	1.0	55.0	44.0	83.43	-.43
			164	80.0	0	44.0	56.0	86.44	-6.44
			165	90.0	0	25.0	75.0	91.00	-1.00
			166	82.0	0	28.0	72.0	90.28	-8.28
			167	80.0	0	18.0	82.0	92.68	-12.68
			168	86.0	0	15.0	85.0	93.40	-7.40
			169	90.0	0	12.0	88.0	94.12	-4.12
			170	98.0	0	10.0	90.0	94.60	3.40
			171	92.0	0	9.0	91.0	94.84	-2.84
			172	78.0	12.0	48.0	40.0	81.04	-3.04
			173	91.0	2.0	20.0	78.0	91.46	-.46
Colorado River near Parker.	Arizona		174	94.0	3.0	70.0	27.0	79.09	14.91
			175	95.0	1.0	37.0	62.0	87.75	7.25
			176	93.0	1.0	59.0	40.0	82.47	10.53
Colorado River near Grand Canyon.	Arizona		177	46.0	30.0	66.0	4.0	70.06	-24.06
			178	87.0	1.0	40.0	59.0	87.03	-.03
			179	88.0	1.0	50.0	49.0	84.63	3.37
			180	87.0	1.0	39.0	60.0	87.27	-.27
			181	84.0	1.0	45.0	54.0	85.83	-1.83
			182	96.0	0	8.0	92.0	95.08	-.92
Colorado River at Lees Ferry.	Arizona		183	97.0	0	23.0	77.0	91.48	5.52
			184	100.0	0	14.0	86.0	93.64	6.36
			185	95.0	0	7.0	93.0	95.32	-.32
			186	92.0	1.0	29.0	70.0	89.67	2.33
			187	98.0	0	20.0	80.0	92.20	5.80

operation. For a more accurate guide to the selection, a duration curve of the reservoir levels can be prepared from the data of the reservoir operation study. This duration curve portrays the functional pattern for study and analysis of the reservoir levels.

Theory and Analysis

The determination of average initial unit weight of deposited sediments in Bureau of Reclamation studies generally requires that sediment concentrations be determined on a percentage by weight basis.

Prediction of the unit weight from these percentages demands use of some type of analytical relation. Wherever possible, such a relation should be consistent with physical laws representing the phenomenon described. In many instances, however, these physical laws may be insufficiently understood or of such a complex nature as to preclude their use. Under these circumstances it may be necessary to use some

form of empirical relation. Three important characteristics of a relation of this type are:

- (1) It should be as simple as consistent with reasonable accuracy.
- (2) It should conform as closely as possible to known data.
- (3) It should predict reasonable values over the entire range of the variables.

In the discussion below, the notation is as follows:

γ = unit weight;

p = fraction by weight of a given component of a sample;

W = constant determined by regression analysis with dimensions of unit weight, applicable to each component;

c, m, s = subscripts denoting clay, silt, and sand, respectively.

Based on simplicity the linear form of equa-

tion used by Lane and Koelzer ⁶ is immediately suggested. It is:

$$\gamma = W_c p_c + W_m p_m + W_s p_s \quad (1)$$

A second simple form is the inverse equation:

$$\gamma = \left(\frac{p_c}{W_c} + \frac{p_m}{W_m} + \frac{p_s}{W_s} \right)^{-1} \quad (2)$$

If, in any given sample of material, each size of particle was separately deposited, equation 2 would describe the physical relation between unit weight and fractional weight of the components correctly. However, due to the random nature of the deposition process, particles of all sizes are intermingled and, for this existing situation, equation 2 must be considered as empirical.

In general, intermixing of the particles of various sizes will produce a denser sediment. This suggests use of a form containing products and powers of the various fractions. Quadratic relations similar to equations 1 and 2, including various combinations of the second degree terms in the variables, were therefore considered.

Still another form, which it was believed might help to account for the intermixture of particle sizes, is the exponential form:

$$\gamma = W_c p_c + W_m p_m + W_s p_s + W p_c^i p_m^j p_s^k \quad (3)$$

in which $i + j + k = 1$.

Equations of each of the above described types were applied to sample populations from the data compiled in table 1. The quadratic and exponential forms gave no significant improvement in the fit of the regression equation to the data selected. In addition, their greater complexity increases the labor involved in their computation and use.

Regression equations of types I and II were derived for all of the data of table 1. For each type of reservoir operation the linear equation gave a closer fit to the data, judged by the correlation coefficient and the standard error of the estimate, than the inverse type. This type of equation was, therefore, adopted for representation of the data. Table 2 summarizes the results from the analysis:

TABLE 2.—Values of coefficients in equation 1 for reservoir types

Type of reservoir operation	Observations Number	Values of coefficients in equation 1			R^1	S^2 Lb./cu. ft.
		W_c	W_m	W_s		
I.....	262	26.1	70.2	105.8	0.839	12.0
II.....	462	27.3	71.4	95.8	.733	14.0
III.....	405	39.6	67.0	104.3	.729	12.2
IV.....	187	59.8	73.1	94.3	.572	10.9

¹ R = correlation coefficient.

² S = standard error of estimate.

Adopted Procedure

In reviewing the previous tabulation by use of equation 1, it is noted that the sand unit weight coefficient varied from 94.3 to 105.8, or a difference of about 11 pounds per cubic foot. It was then decided that the data of all 1,316 samples be computed in a single run for the purpose of obtaining a representative value of the unit weight coefficient for sand under all types of reservoir operation. These computations resulted in a unit weight coefficient of about 97 pounds per cubic foot for sand. This value was adopted for use in the investigation.

A further inspection of the tabulated values showed a slight decrease in the unit weight coefficient for silt for the type III reservoir operation. This would be contrary to the normal manner by which the sediments physically compact in a reservoir; the value, therefore, was revised upward to 72 pounds per cubic foot. Unit weight coefficients for clay appeared consistent with the normal compaction pattern of sediment accumulation for each type of reservoir operation.

Based upon the preceding revision, a different set of values was obtained for each unit weight coefficient. Table 3 summarizes the final values rounded to the nearest pound as determined from this investigation.

The values listed above would consequently revise the correlation coefficients and standard error of estimates as indicated, although neither is greatly changed from those listed in the previous tabulation.

Using the coefficients from the above tabulation, the unit weight of each sample was computed and tabulated in table 1 under the column headed "Computed unit weight." The individual deviations, δ , were also determined from the difference between the observed and computed unit weights. Figures 1 through 4 show graphically the comparison of these observed and computed unit weights for each type of reservoir operation.

The enveloping lines shown in these figures represent the ranges of plus or minus one standard error of estimate and 1.645 times the standard error of estimate indicated, respectively, by the long dashed and short dashed

⁶ See footnote 4.

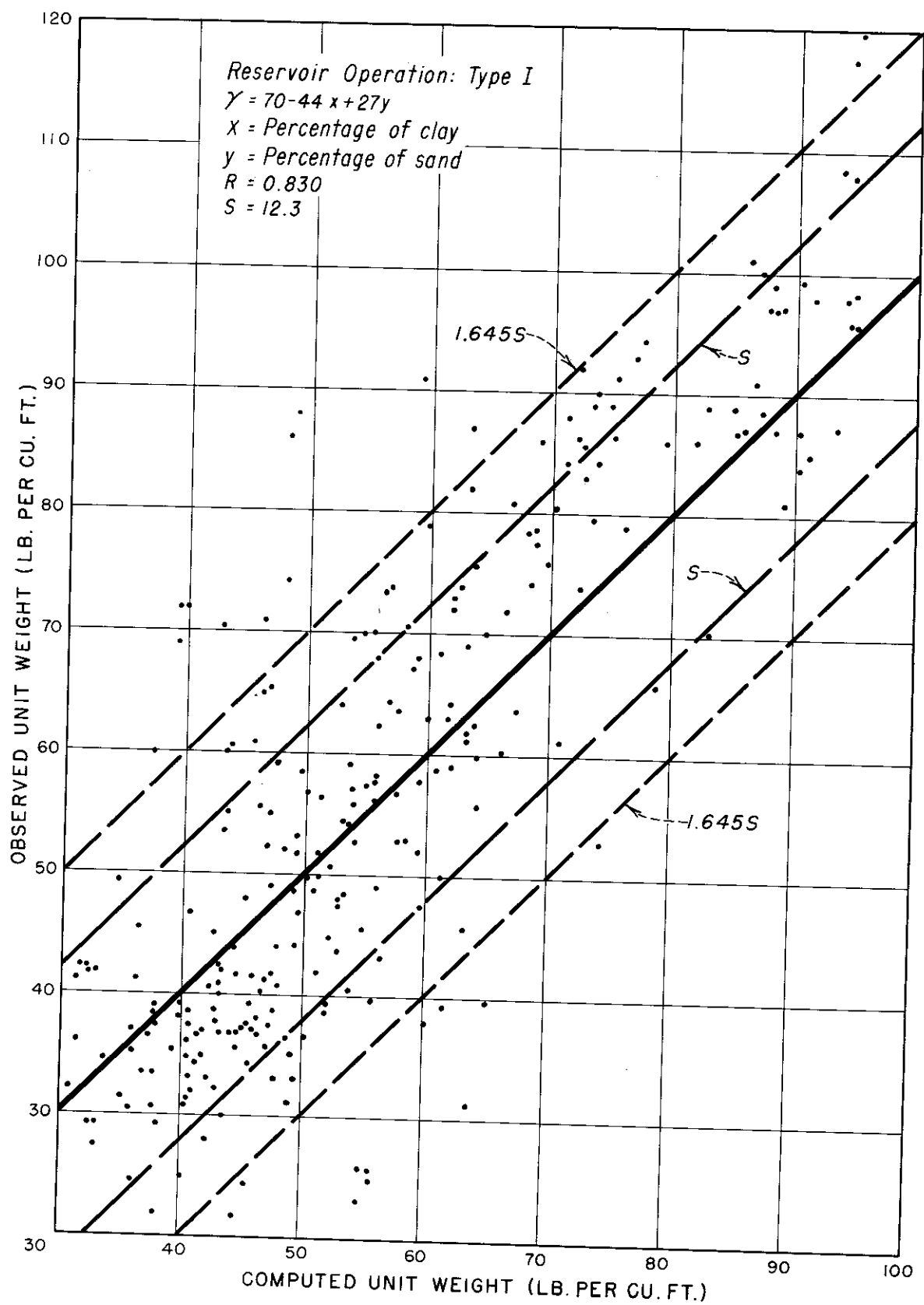


FIGURE 1.—Correlation of observed vs. computed unit weights. Reservoir operations: Type I.

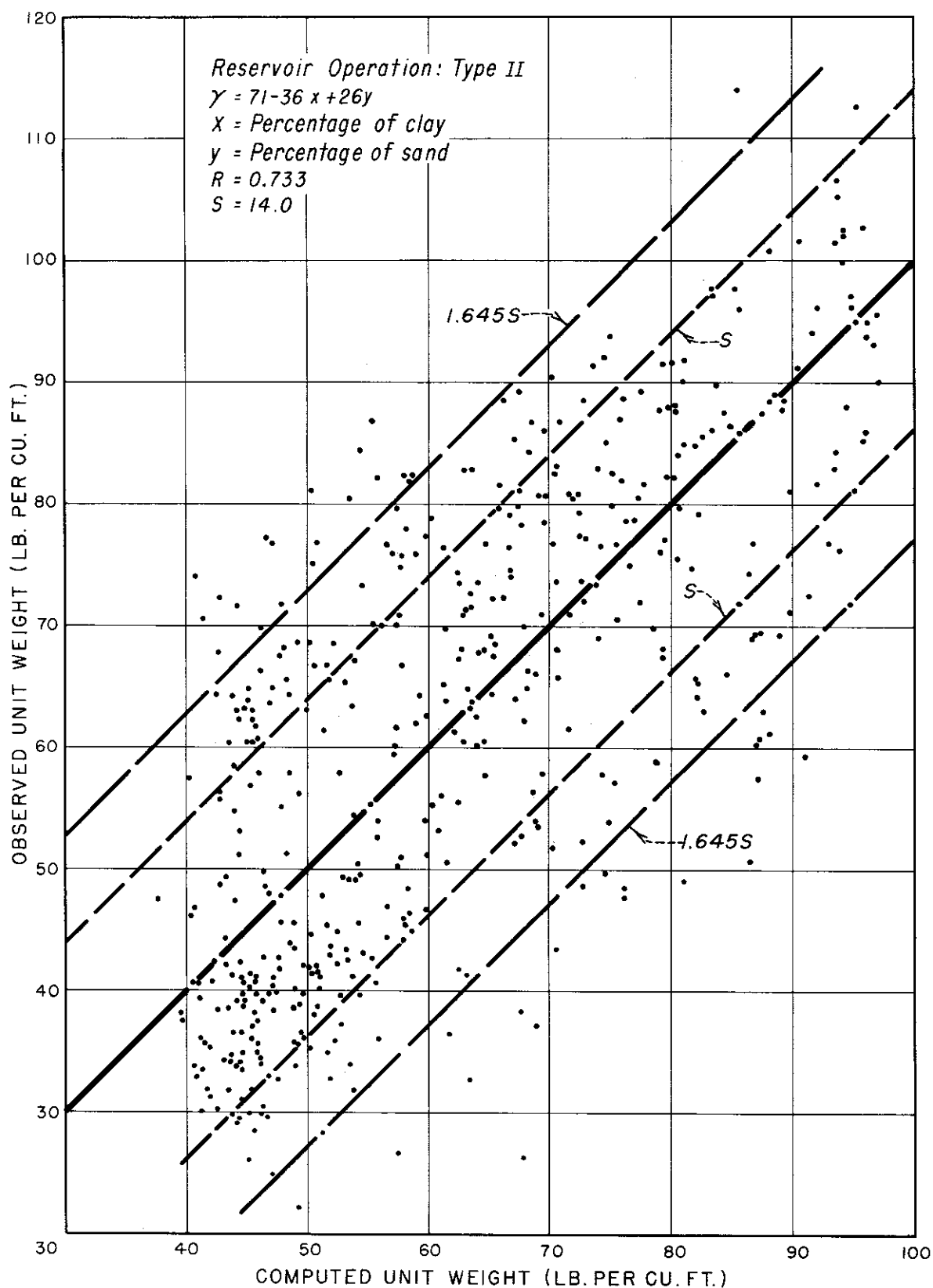


FIGURE 2. — Correlation of observed vs. computed unit weights. Reservoir operation: Type II.

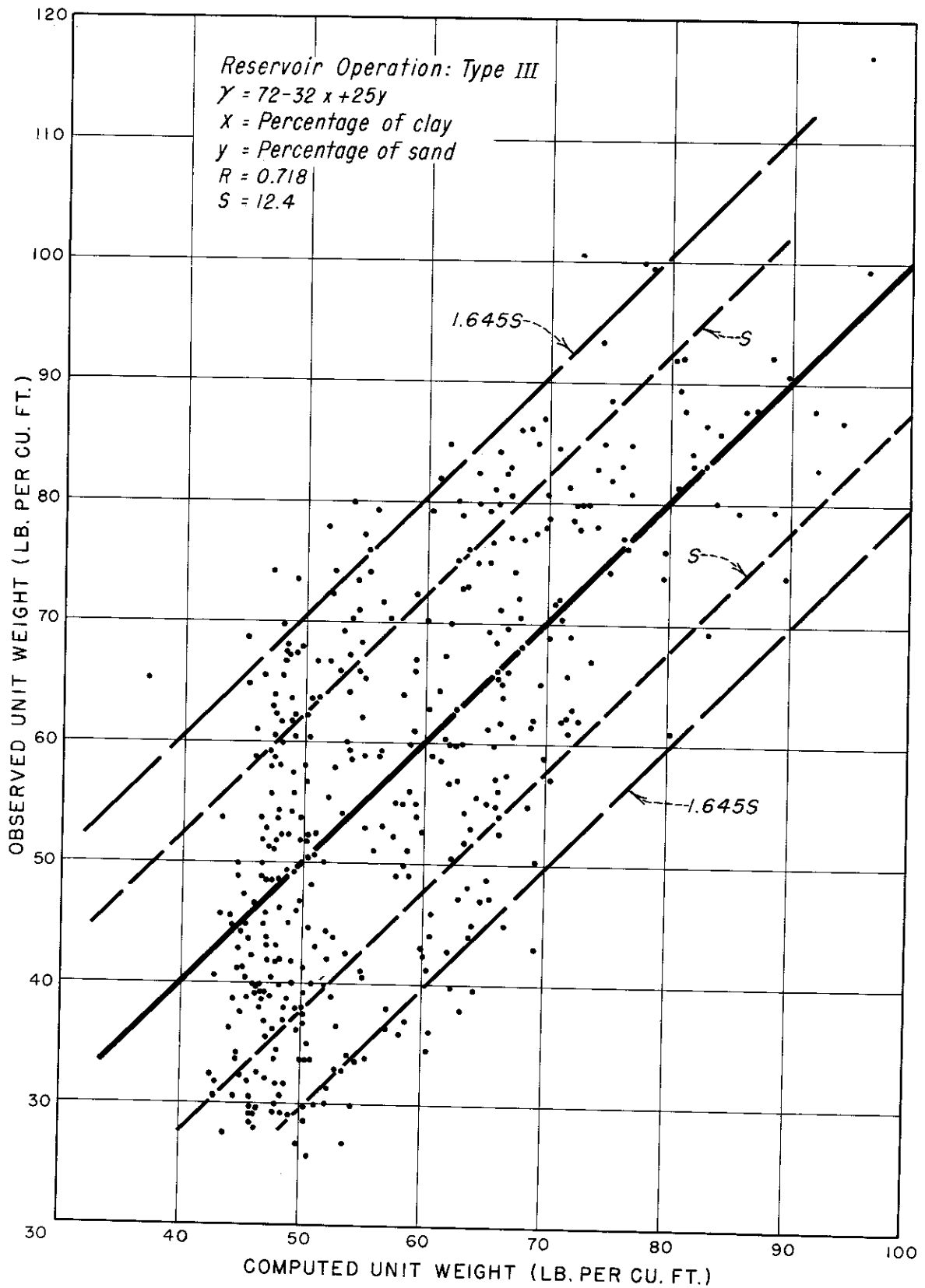


FIGURE 3. — Correlation of observed vs. computed unit weights. Reservoir operation: Type III.

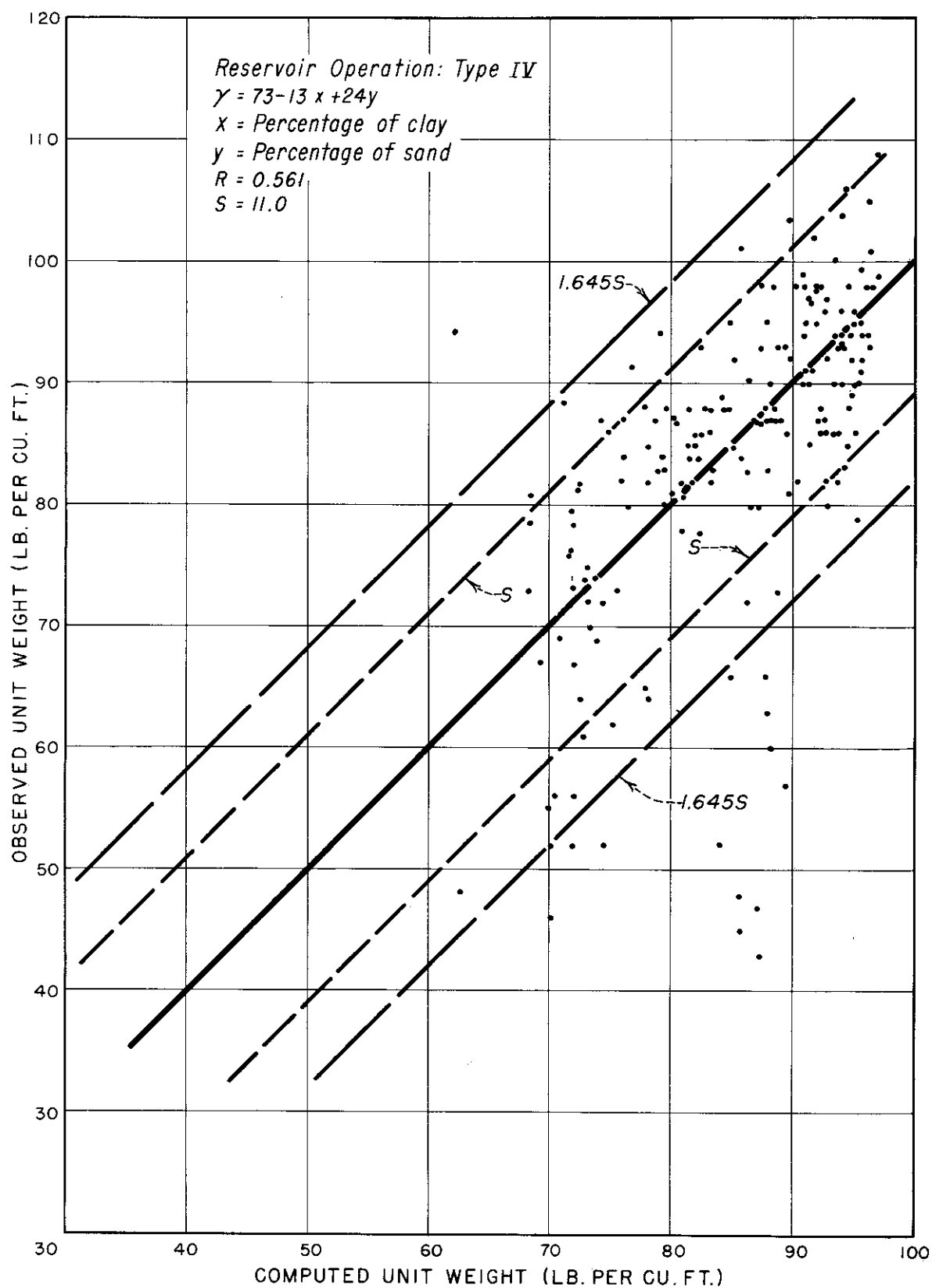


FIGURE 4. — Correlation of observed vs. computed unit weights. Reservoir operation: Type IV.

TABLE 3.—*Modified values of coefficients in equation 1 for reservoir types*

Type of reservoir operation	Observations	Values of coefficients in equation 1			R	S
		W_c	W_m	W_s		
	Number					Lb./cu. ft.
I.....	262	26	70	97	0.830	12.3
II.....	462	35	71	97	.733	14.0
III.....	405	40	72	97	.718	12.4
IV.....	187	60	73	97	.561	11.0

lines. One standard error of estimate would embrace about 68 percent of the residuals or differences between computed and observed values while about 90 percent would line within the range of 1.645 times the standard error.

The unit weight as determined from values of the above tabulation would be considered as the initial unit weight. No attempt has been made in this investigation to correct for the compaction of sediments as was done by Miller. This is a special phase and will be considered separately in a future study.

Sample Computation

An example of the computational procedure to determine the initial unit weight will now be given. The procedure simply entails the application of equation 1 for a given set of field data.

Given:

From the size analysis, it is found that the sediments are composed as follows:

Sediment type	Percentage
Clay	23
Silt	40
Sand	37

⁷ See footnote 2.

⁸ See footnote 1.

⁹ HEINEMANN, H. G. VOLUME-WEIGHT OF RESERVOIR SEDIMENT. Amer. Soc. Civil Engin. Jour. Hydraulics Div. 88 (No. HY5): 181-197.

Reservoir operation: Type I.

Compute γ :

$$\begin{aligned}\gamma &= W_c p_c + W_m p_m + W_s p_s \\ &= (26) (0.23) + (70) (0.40) + (97) \\ &\quad (0.37) \\ &= 5.98 + 28.00 + 35.89 = 69.87 \text{ lb./cu. ft.}\end{aligned}$$

This can be rounded to 70 pounds per cubic foot for practical use and for use as initial unit weight of sediment deposits.

Summary and Conclusions

It is concluded from this investigation that the unit weights can be computed on the basis of the regression equations derived from the analysis of the 1,316 samples compiled. This study showed better statistical results in deriving empirically the unit weight coefficients by equation 1 than by equation 2. Because of the greater number of samples analyzed, these coefficients would supersede those listed in Miller's⁷ and the Lane and Koelzer's⁸ studies. It is suggested that the coefficients derived herein be used in future investigations where initial unit weight values need to be determined.

Future studies should be aimed at quantitatively determining the various factors affecting the unit weight of deposited sediments. This can be more readily accomplished by conducting sedimentation surveys of the type Heinemann⁹ made for Sabetha Lake.

IMPROVED VOLUMETRIC SURVEY AND COMPUTATION PROCEDURES FOR SMALL RESERVOIRS

[Paper No. 83]

By H. G. HEINEMANN and V. I. DVORAK, *hydraulic engineers, Soil and Water Conservation Research Division, Agricultural Research Service*

Abstract

Hundreds of small reservoirs are surveyed every year to obtain information on storage capacity, sediment volumes, or for other purposes, at considerable time, effort, and expense for the associated field work and office computations. The final results from these surveys, however, are not always comparable or the data as useful as desired, because of poor procedures or techniques.

In a quest for the best survey and computa-

tion methods consistent with available resources, four small reservoirs were surveyed in great detail. Different surveys were imposed on these detailed bases and numerous computation techniques were used in the office calculations.

It was found that a field determination of the emergency spillway contour and a selected lower contour, together with data from sufficient ranges properly located (parallel where possible) provide adequate and dependable information. The integrimeter is a time-saving area-measuring tool that can be used to advantage.

Experience elsewhere and this particular study show that the stage-area curve method is the most direct, simple, accurate, and uniformly adaptable way to determine the capacity of a reservoir. Volumes of sediment deposits can best be obtained from capacity differences.

Introduction

Many small reservoirs are surveyed every year by Federal, State, and local agencies to obtain information on reservoir storage capacity, amount of deposited sediment, sediment distribution, and other factors. The need for good volumetric data in the design of water control and utilization projects has been so prominently emphasized in the past that it needs no further discussion here.

Considerable time, effort, and money are expended in the field and office work accompanying these reservoir surveys. However, the final results are not always as dependable as might be desired or as comparable with other reservoir

data, because of the use of poor procedures. This situation should be improved.

This study was undertaken to determine the best procedures, consistent with the usually available resources, for determining the volume (remaining storage capacity) of a reservoir. Emphasis in this paper is on reservoir volumes rather than sediment volumes. Where sediment volumes are also desired, as pointed out in the paper, they can be obtained as the difference in remaining reservoir storage capacity at different dates. In the case of reservoirs without prior volume surveys, the measurements of sediment thicknesses can be made with a spud¹ or other acceptable device.

Volumetric Survey Procedure

The four reservoirs used as a basis for this study were selected because they were surveyed in great detail and they provided a variety of reservoir topography. They are relatively small, ranging in capacity from 29 to 70 acre-feet (figs. 1 to 4). The spacing of many of the cross sections near the dams is 25 feet, but spacings of 50 feet or more were used in the upstream parts

¹ EAKIN, H. M. (rev. by Brown, C. B.) SILTING OF RESERVOIRS. U.S. Dept. Agr. Tech. Bul. 524, 168 pp. 1939.

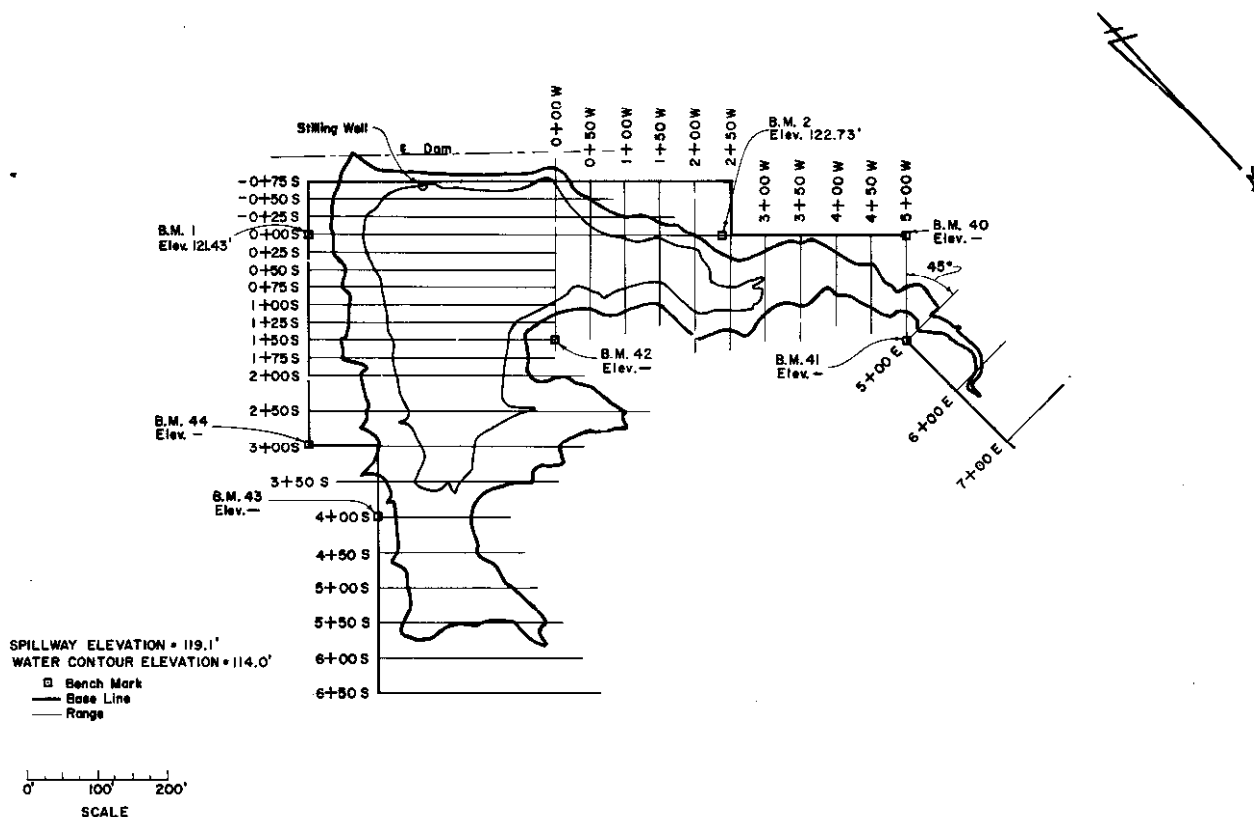


FIGURE 1. — Matt Hafner Reservoir, Newell, S. Dak., sedimentation survey, September 3, 1959.

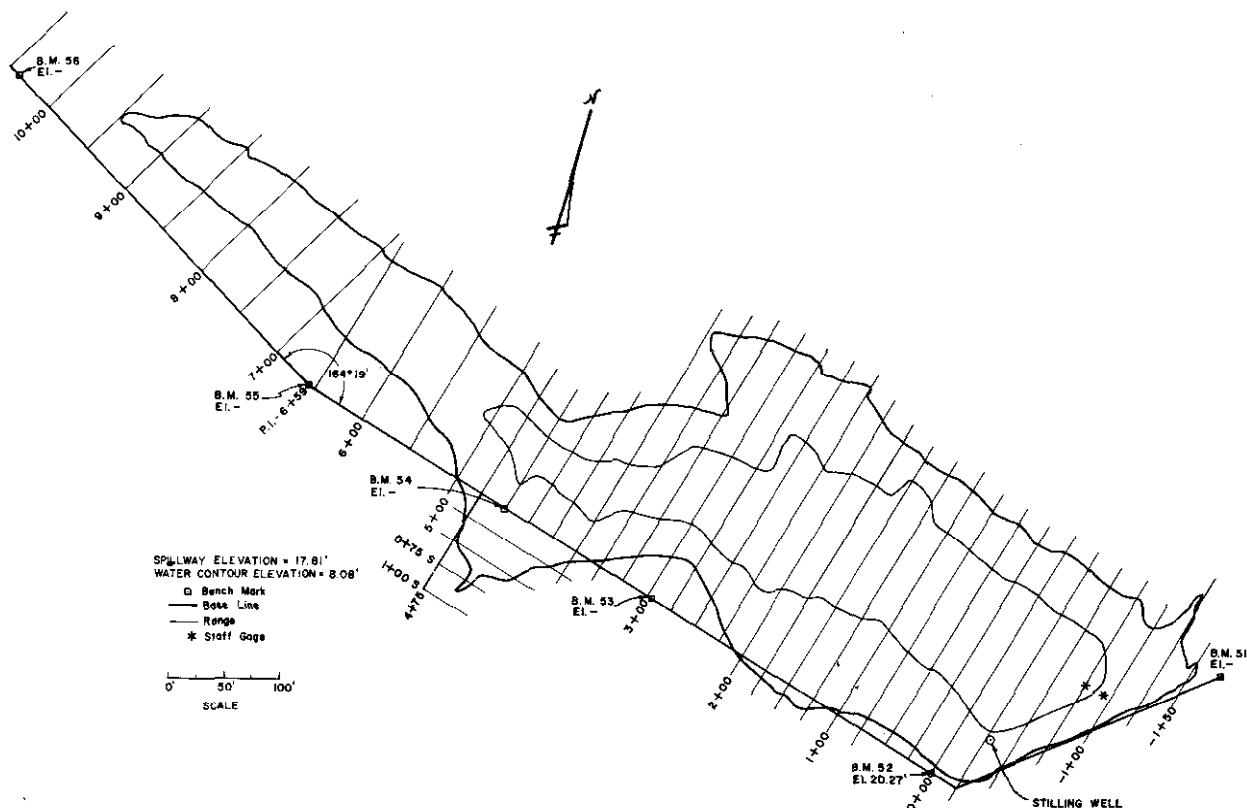


FIGURE 2. — Bertha Armour Reservoir, Newell, S. Dak., sedimentation survey, August 1958.

of the reservoirs. The survey method used on these four reservoirs proved very effective, and was used as a guide for the following suggested survey procedures.

Figure 5 is a drawing of a hypothetical reservoir showing many of the shapes encountered in the field. It will be used with the following discussion, which is applicable to the survey of a reservoir several years old and on which no survey base was established at the time the dam was completed.

Preliminary Field Work

Before actually starting field work, the engineer should study the topography in the surrounding area, especially that directly downstream from the dam. The slopes leading into the reservoir valley should be examined to determine if they are continuous or if a bench, terrace, or other irregularity exists. The presence of such topographic features will require consideration and adequate delineation during the fieldwork. Also, it is necessary to decide upon the method of computing the volume, since the field survey data required are partially dependent on the computational procedure.

Where possible, a measured base line should

be established along one side of the reservoir, and this should generally be parallel to the main valley. Such a base line requires few permanent monuments and provides for greater accuracy in the survey itself.

Experience has shown that a reservoir contour map can be drawn adequately from a field determination of the upper spillway contour, a selected lower contour, and data from a sufficient number of ranges properly located. A range is simply a fixed line across a reservoir, along which an adequate number of elevations are determined to permit the establishment of the topography. The proper location of these ranges is facilitated if the plane table work locating contours is completed first.

Information as to the locations of the spillway contour (upper spillway—principal or emergency, depending on the type of structure) and another selected lower contour is desired, because these contours provide excellent horizontal control. The spillway contour should be mapped on a plane table sheet by using alidade and level equipment. This work proceeds quite rapidly if an engineer's level is used to insure that the rod is on the contour while the contour

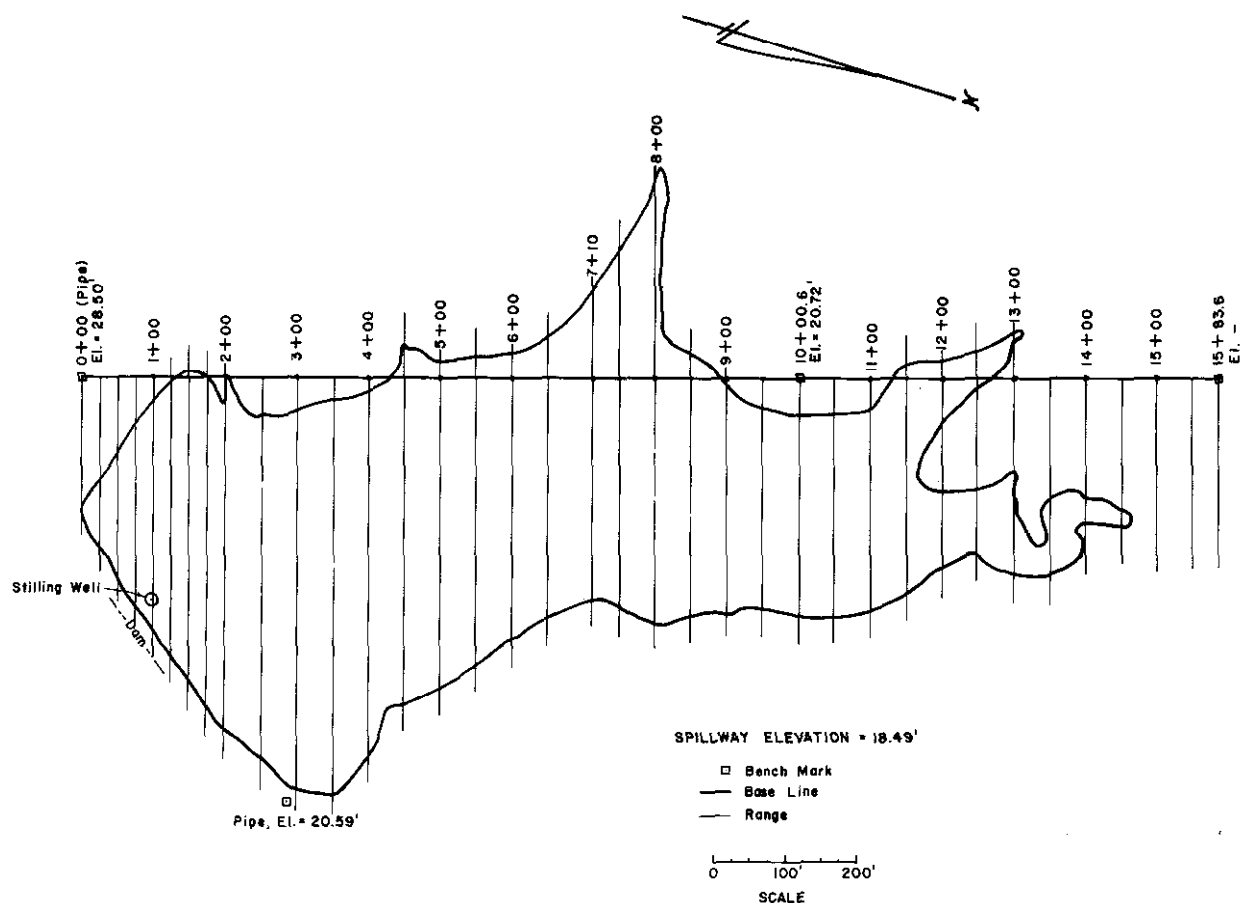


FIGURE 3. — Al Olsen Reservoir, Newell, S. Dak., sedimentation survey, July 1958.

is being located on the plane table sheet. A selected lower contour is desirable because it provides information on the changes in the land slopes between the ranges, and datum from it provides a strategic point on a stage-area curve. This selected lower contour is more valuable if it is located about one-third of the total reservoir depth down from the spillway. If the water stage is at this approximate elevation, the contour can be mapped rapidly, inasmuch as the level is not needed. If the reservoir is full, or nearly so, this lower contour is usually omitted.

Ranges

Ranges provide excellent vertical controls and should be located to yield maximum data for drawing contours. Ranges should be placed generally perpendicular to the valley and perpendicular to the base line, if possible. This greatly facilitates the computations. The ranges should be extended so that the maximum elevations are at least 3 feet above the highest spillway. It is important to have good range coverage so

that the topography can be adequately portrayed.

If the exact topography of the upstream face of the dam is not known, at least one range should be located perpendicular to the axis of the dam to establish the presence or absence of a berm and also to locate the toe of the fill. If only a minimum number of ranges is used, one range must be located across the reservoir along the upstream toe of the dam.

Figure 5 shows a suggested range layout for a small reservoir. Most of the ranges are parallel to each other and perpendicular to the base line. Other ranges were placed in specific locations to facilitate obtaining special information.

Elevations along the ranges below the water level are determined by sounding with a sounding pole or bell, or other means. If the underwater topography is quite uniform, elevation determinations can be widely spaced. However, if the terrain is rough, the determinations should be numerous so that good topographic maps can be plotted. If a sounding bell is used, the graduations on the line should be checked

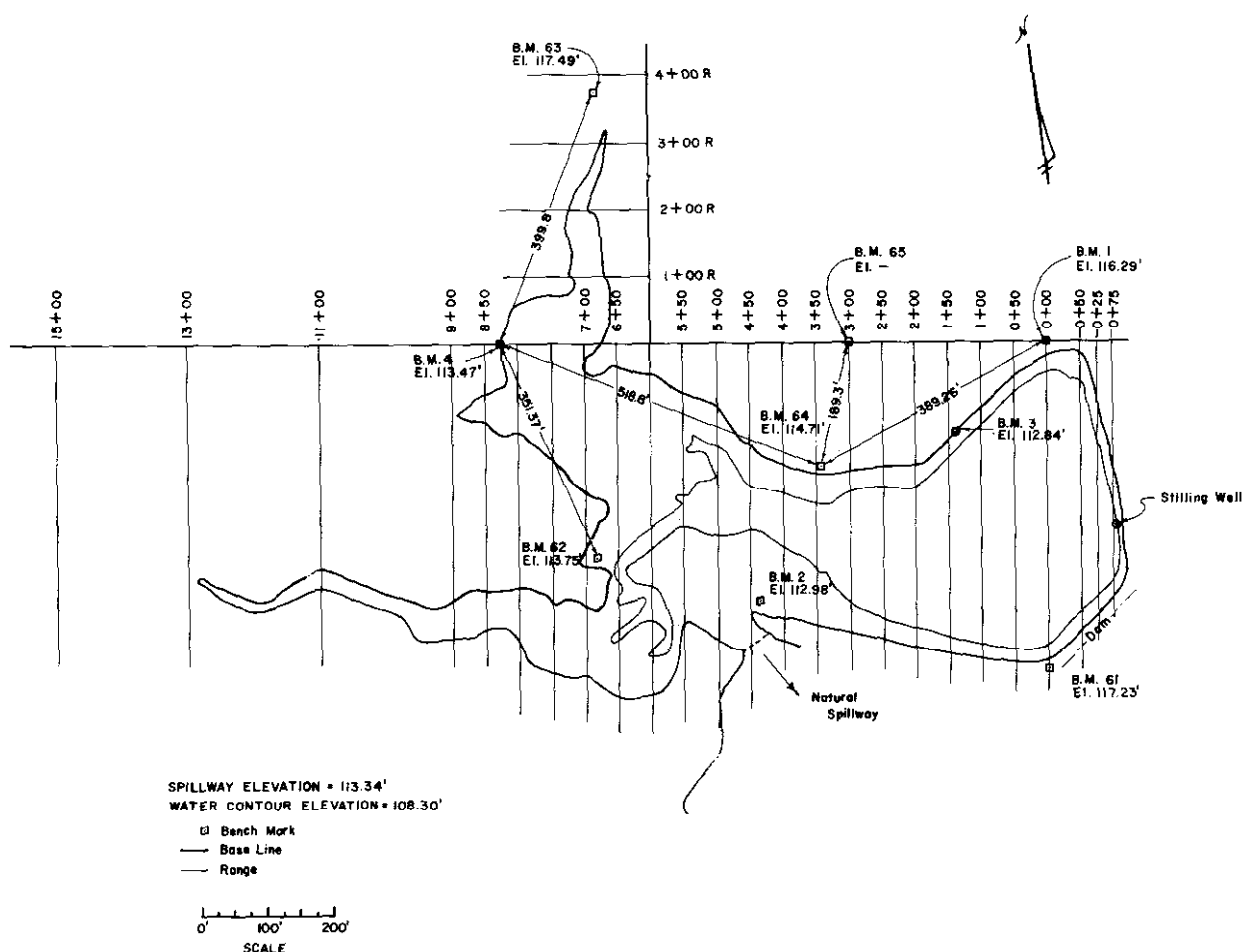


FIGURE 4. — Mitchell Reservoir, Newell, S. Dak., sedimentation survey, May 1960.

frequently for accuracy. Gottschalk² has described a system for marking the line. Elevations above the water level should be determined by standard level survey procedures.

If other data are to be obtained during the survey, such as the volume-weight of sediment, they should be obtained along the ranges. Accurate locations, both as to distance from range end and depth of sample, should be recorded.

In connection with all of the foregoing, it is more practical to obtain more than adequate, rather than insufficient data. It takes very little field time to obtain information along a few additional ranges, and such data may prove highly desirable during the computations.

Methods of Calculations

Numerous computational techniques were used to determine the reservoir capacities of

² GOTTSCHALK, L. C. SOUNDING AND SPUDDING LINES. U.S. Soil Conservation Service Tech. Letter Sed-19, dated August 1, 1948.

the four reservoirs studied. These techniques were used with the following amounts of data: (1) the complete detailed reservoir surveys; (2) parts of the detailed surveys; and (3) imposed surveys developed from the detailed bases. The computational techniques can be grouped into two basic approaches—contour areas and cross-sectional areas.

Contour Area Methods

The contour area approach is dependent on contours to reflect actual topography adequately. Experience has shown that topographic maps with 2-foot contour intervals are adequate for the size of reservoirs represented by this study. This approach can be used with the following four methods: (1) Stage-area curve; (2) modified prismoidal; (3) Simpson's rule; and (4) average contour area.

Stage-area curve method

The stage-area curve method requires the plotting of a well-defined stage-area curve. If de-

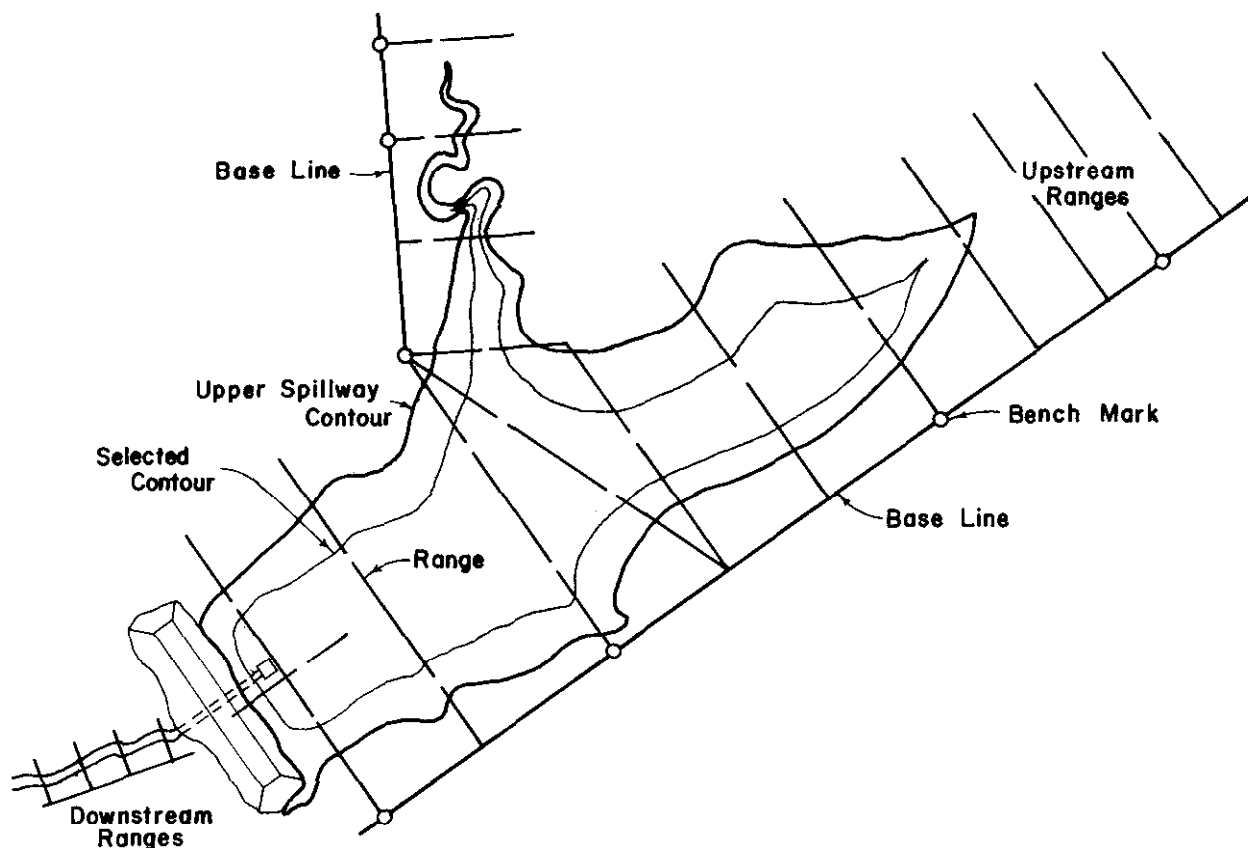


FIGURE 5. — Suggested range layout for a typical reservoir.

sired for Reservoir Sedimentation Data Summary forms,³ such curves can be developed for various segments of the reservoir. The area between such a curve and corresponding selected elevations on the stage axis represents the reservoir capacity between the selected elevations. As an example, the capacity between elevations 112 and 114 (fig. 6) is represented by the shaded area. This can be planimetered and converted directly to capacity.

Modified prismoidal method

Figure 7 shows the equation used in the modified prismoidal method. The average of three areas is multiplied by the contour interval. The middle area is an approximation and an attempt to duplicate the midpoint of the segment on the stage-area curve. The accuracy of the results decreases with increased curvature in the stage-area curve.

In this method, the symbols represent the

following:

- V = capacity, in acre-feet;
- L = contour interval, in feet;
- A = area of lower contour, in acres;
- B = area of higher contour, in acres.

Simpson's rule

Simpson's rule has never been mentioned in the reservoir survey literature, but it can be used to compute the capacity. A requirement is that the reservoir must be divided into an even number of segments. The general equation is

$$V = \frac{1}{3} h [A_0 + A_n + 4(A_1 + A_3 + \dots + A_{n-1}) + 2(A_2 + A_4 + \dots + A_{n-2})]$$

where

- V = capacity, in acre-feet;
- A = area of contour or cross section, in acres; and
- h = interval spacing between contours or cross sections, in feet.

Average contour area method

This method consists of averaging the upper and lower contour areas and multiplying by the contour interval. The summation of such de-

³ SUBCOMMITTEE ON SEDIMENTATION, INTER-AGENCY COMMITTEE ON WATER RESOURCES. SUMMARY OF RESERVOIR SEDIMENTATION SURVEYS MADE IN THE UNITED STATES THROUGH 1953. Sedimentation Bul. 6: Appendix. April 1957.

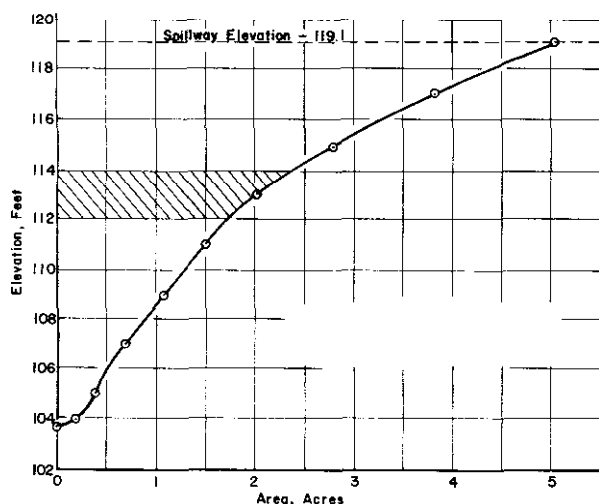


FIGURE 6.—Matt Hafner Reservoir stage-area curve.

terminations approximates the capacity of the reservoir.

Cross-Sectional Area Methods

The cross-sectional area approach depends primarily on cross-sectional areas (of ranges) in volumetric computations. The location of ranges, therefore, must be selected carefully to show the topography. This approach can be used with the following four methods: (1) Eakin's range end formula; (2) cross-sectional area vs. distance from dam—curve; (3) Simpson's rule, using the cross-sectional area method; and (4) average end area.

Eakin's range end formula

In this volumetric formula, both cross-sectional and surface areas are used, as explained by Eakin and Brown,⁴ and also by Gottschalk.⁵ It is based on the prismoidal formula and is shown in figure 8 with a segment illustration.

The symbols are:

V =capacity, in acre-feet;

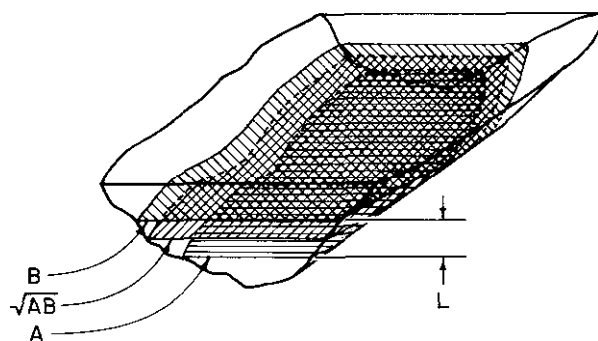
A' =quadrilateral area—the area, in acres, of the quadrilateral formed by connecting the points of intersection of the ranges with crest contour;

A =lake area of the segment, in acres;

E =range cross-sectional area, in square feet;

W =width of the range at crest elevation, in feet;

h =the perpendicular distance, in feet, from one range to the shore line at the other range;



$$V = \frac{L}{3} (A + \sqrt{AB} + B)$$

FIGURE 7.—Modified prismoidal formula.

h_3 =the perpendicular distance, in feet, from the range on a tributary to the junction of the tributary with the main stream.

The subscripts shown in the figure identify these quantities with the respective ranges of the segment.

Reservoirs having irregular-shaped segments may require as many as five different variations of this formula to compute the total capacity. A standard form is available for uniform and orderly calculation of the basic formula. A template has been devised in conjunction with this form to speed the calculations and help prevent errors.

Cross-sectional area vs. distance from dam—curve method

On a graph showing range cross-sectional areas plotted versus distances from the dam, the capacity is represented by the area between the curve and the "distance from the dam" axes. The accuracy is dependent on the ade-

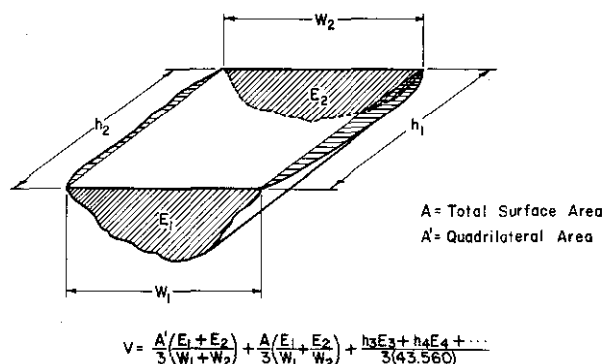


FIGURE 8.—Eakin's range formula.

⁴ See footnote 1.

⁵ GOTTSCHALK, L. C. MEASUREMENT OF SEDIMENTATION IN SMALL RESERVOIRS. Amer. Soc. Civil Engin. Proc., Hydraul. Div. 77 (55): 11 pp. 1951.

quacy of the defined curve. Figure 9 shows such a curve.

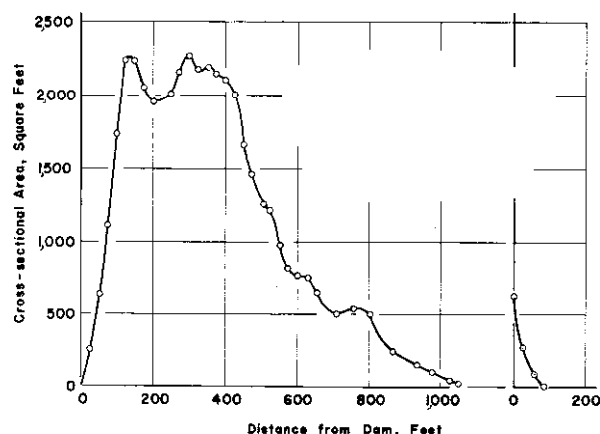


FIGURE 9. — Bertha Armour Reservoir cross-sectional area versus distance from dam.

Simpson's rule

The use of Simpson's rule, with the cross-sectional area approach, was made possible by the many evenly spaced parallel ranges. In

several instances, it was necessary to use another cross-sectional area method to compute the volume of the final odd segment.

Average end area method

The average end area method requires a determination of the average of the cross-sectional areas. This average is then multiplied by the total length of the reservoir to obtain the capacity.

Sediment Volume

The easiest and most practical method of computing the volume of sediment deposited in a reservoir between certain survey dates is by determining the difference in capacities on the two dates. In the case of a reservoir without satisfactory prior volume surveys, sediment thicknesses can be determined with a spud or other suitable device and the necessary information obtained for establishing the original contours or cross-sectional areas of the reservoir. In this latter case, the sediment volume is the difference between the original reservoir capacity and that remaining at the date of survey.

Short Cuts

Time-saving devices and procedures were used whenever possible. All measurements on the stage-area curve and contour area measure-

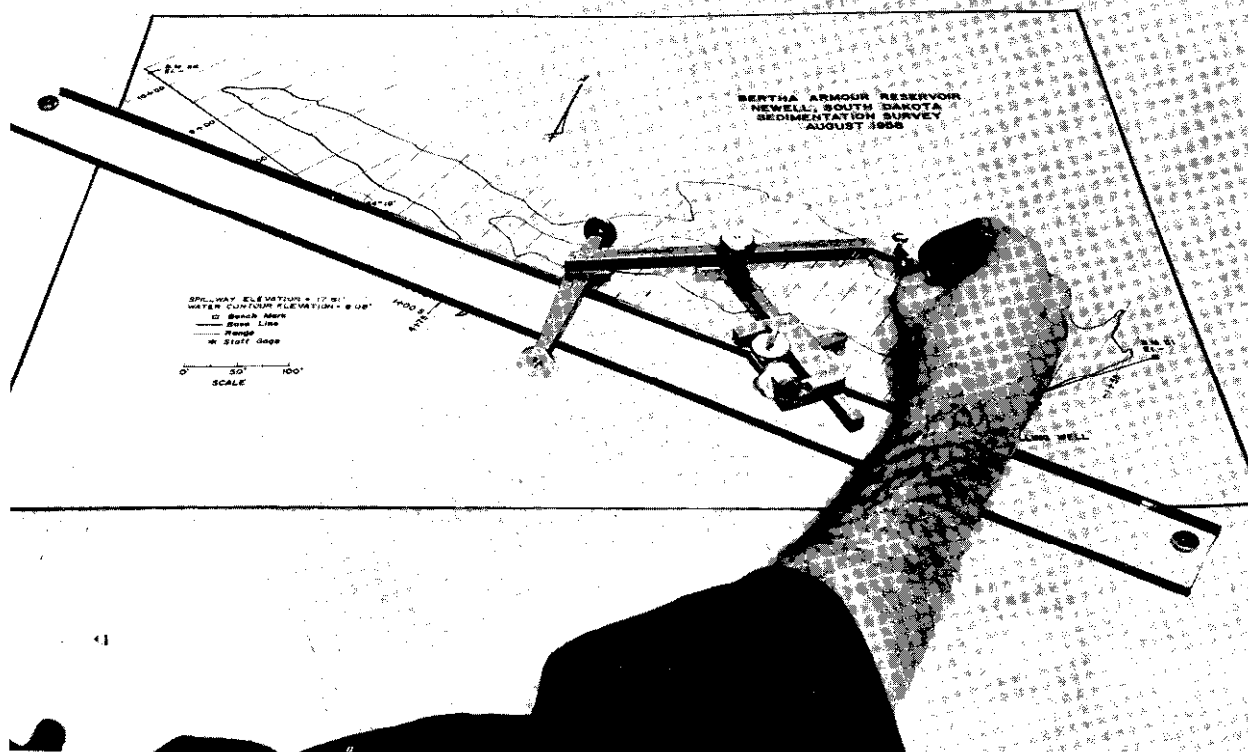


FIGURE 10. — Integrimeter.

ments when the ranges were parallel to each other were made with the integrimeter. Figure 10 shows this instrument. Its use saves a great deal of time. It determines the integral value from point to point on a curve while tracing the curve; it determines directly the area below any portion of a traced curve. It can also be used as a planimeter. It has another advantage in that the measuring wheel runs on a special surface at all times, minimizing differential wheel slippage.

A short-cut method for determining cross-sectional areas directly from reduced field notes has been developed by Mr. Dvorak. This method is a programming of a general trapezoidal formula for determining cross-sectional areas on an electric calculator having a single keyboard. This method saves from one-third to two-thirds the normal time required to determine such areas. Furthermore, a check is made of all entries simultaneously with the computations. Figure 11 shows the tabulation forms used with Dvorak's method. It shows the simplicity of the tabulation involved in the computation and its check for each of Dvorak's two methods.

SUMMARY FORM - CROSS SECTIONAL AREAS
COMPUTED BY THE ELEVATION METHOD

Reservoir _____
State _____
Survey Date _____

Computer _____ Date _____

Range or Cross Section No. _____

Calculation Check: $\sum(\text{Distance} + \text{Elevation}) =$ _____

(Distance _____) $\times$ (Elev. Spwy. _____) = _____

minus Double Area (_____) $\frac{1}{2} =$ _____

Cross Sectional Area (Ft.²) = _____

SUMMARY FORM - CROSS SECTIONAL AREAS
COMPUTED BY THE DEPTH METHOD

Reservoir _____
State _____
Survey Date _____

Computer _____ Date _____

Range or Cross Section No. _____

Calculation Check: $\sum(\text{Distance} + \text{Depth}) =$ _____

Double Area = _____

Cross Sectional Area (Ft.²) = _____

FIGURE 11.—Illustration of form used in computing cross-sectional areas in the elevation and depth methods (after Dvorak's shortcuts).

Results

The field survey and the office computations are used together to obtain reservoir capacity. The best type of computation procedure is of little value without an adequate field survey. The fieldwork should be oriented with the computation method so that the essential information is always obtained.

The results of the various computations applied to the data for the small reservoirs are shown in tables 1 to 4. A study of these tables

TABLE 1.—Summary of capacity computations—
Al Olsen Reservoir

Methods of computation	Contour interval	Type of survey	Remaining capacity
	<i>Feet</i>		<i>Acres-feet</i>
Stage-area	2	34 parallel ranges . . .	68.06
	4	34 parallel ranges . . .	68.01
	2	4 parallel ranges, 12-foot contour	68.56
	2	7 skewed ranges, 12-foot contour	69.16
Modified prismoidal	2	34 parallel ranges . . .	67.69
	4	34 parallel ranges . . .	67.36
	2	4 parallel ranges, 12-foot contour	68.74
	2	7 skewed ranges, 12-foot contour	69.33
Eakin's range formula		34 parallel ranges . . .	68.23
		4 parallel ranges . . .	72.43
		7 skewed ranges . . .	64.07
Simpson's Rule (range cross-sectional area):	2	34 parallel ranges . . .	67.81
		34 parallel ranges . . .	67.69
		29 parallel ranges . . .	67.54
Average contour area	2	34 parallel ranges . . .	68.08
	4	34 parallel ranges . . .	68.80
	2	4 parallel ranges, 12-foot contour	69.11
	4	4 parallel ranges, 12-foot contour	70.49
	2	7 skewed ranges, 12-foot contour	69.71
	4	7 skewed ranges, 12-foot contour	70.93
Cross-sectional area versus distance from the dam.		34 parallel ranges . . .	68.34

reveals that when all the available survey data (all information from ranges spaced 25 or 50 feet) were used with each of the methods of computation, the capacities were almost identical. With a great deal of data, any method is adequate. Some methods are a little cruder than others, and this is generally shown in the results.

The stage-area method result was considered to be the most accurate and was used as a basis for other comparisons. This method is more direct than others and does not use approximation formulas.

TABLE 2.—*Summary of capacity computations—
Bertha Armour Reservoir*

Methods of computation	Contour interval	Type of survey	Remaining capacity
	<i>Feet</i>		<i>Acre-feet</i>
Stage-area	2	39 parallel ranges . . .	26.14
	2	6 parallel ranges, 8.08-ft. contour . . .	25.96
Modified prismoidal	2	39 parallel ranges . . .	26.06
	2	6 parallel ranges, 8.08-ft. contour . . .	25.67
Eakin's range formula		39 parallel ranges . . .	26.08
		6 parallel ranges . . .	24.15
Simpson's Rule (range cross-sectional area)	2	39 parallel ranges . . .	26.18
		39 parallel ranges . . .	26.07
Average contour area	2	39 parallel ranges . . .	26.14
	2	6 parallel ranges, 8.08-ft. contour . . .	25.78
Average end area		39 parallel ranges . . .	26.19
Cross-sectional area versus distance from the dam		39 parallel ranges . . .	26.20

TABLE 3.—*Summary of capacity computations—
Matt Hafner Reservoir*

Methods of computation	Contour interval	Type of survey	Remaining capacity
	<i>Feet</i>		<i>Acre-feet</i>
Stage-area	2	34 parallel ranges . . .	29.66
	2	9 parallel ranges 14-ft. contour . . .	29.58
	4	9 parallel ranges, 14-ft. contour . . .	29.81
	2	7 skewed ranges, 14-ft. contour . . .	29.73
	4	7 skewed ranges, 14-ft. contour . . .	29.76
Modified prismoidal	2	34 parallel ranges . . .	29.63
	2	9 parallel ranges, 14-ft. contour . . .	29.60
	2	7 skewed ranges, 14-ft. contour . . .	29.09
Eakin's range formula		34 parallel ranges . . .	29.63
		9 parallel ranges . . .	30.74
		7 skewed ranges . . .	30.26
Simpson's Rule (range cross-sectional area)		34 parallel ranges . . .	29.70
Average contour area	2	9 parallel ranges, 14-ft. contour . . .	29.74
	2	7 skewed ranges, 14-ft. contour . . .	29.71

When a study was made of the results obtained by using the data from a small number of parallel ranges, the stage-area method capacities more nearly approached the volumes determined by using all the data. This was followed by the results from other methods based on con-

TABLE 4.—*Summary of capacity computations—
Mitchell Reservoir*

Methods of computation	Contour interval	Type of Survey	Remaining capacity
	<i>Feet</i>		<i>Acre-feet</i>
Stage-area	2	26 parallel ranges . . .	38.31
Modified prismoidal	2	26 parallel ranges . . .	38.41
Eakin's range formula		26 parallel ranges . . .	38.19
Simpson's Rule (range cross-sectional area)		26 parallel ranges . . .	38.59
Average contour area	2	26 parallel ranges . . .	38.71
Average end area		26 parallel ranges . . .	38.24

tour areas and then by those involving cross-sectional areas. Normally, more than four ranges are needed in determining the remaining capacities. Figure 12 shows the few selected parallel ranges and the skewed ranges that were used on the Al Olsen Reservoir for comparative purposes.

When only the data along a few skewed ranges were used in the computations, the stage-area method again provided the best results. Figure 12 shows a suggested range layout for a reservoir on which parallel ranges are not practical. Note that they were so selected that, in conjunction with the selected contours, they furnish a maximum of topographic data.

Different individuals surveying this reservoir might select different range layouts and thus obtain different data for the computations. These would result in different reservoir capacity values. They might or might not be as good as those shown in table 1. Not all possible computation methods were tried with the data from the few ranges shown in figure 12 because some were no longer applicable.

A selected lower contour was used to provide additional data on topography. This information was useful with the contour approach but such supplemental information cannot be inserted into the cross-sectional area approach.

Accurate records of the time required for each computation method were not maintained. However, it is believed that the desired remaining capacity, stage-storage and stage-area curves, and other information useful on the Reservoir Sedimentation Data Summary forms, can be obtained in the shortest time with the stage-area method. It is quicker than the other methods using the contour approach because formula calculations are not involved. Experience with the various methods also indicates that the stage-area method is faster than the methods employing cross-sectional areas. The stage-area method is also more uniformly adaptable than most of the other methods. It does not require separate formulas for various segments of the lake, nor does it require an even number of con-

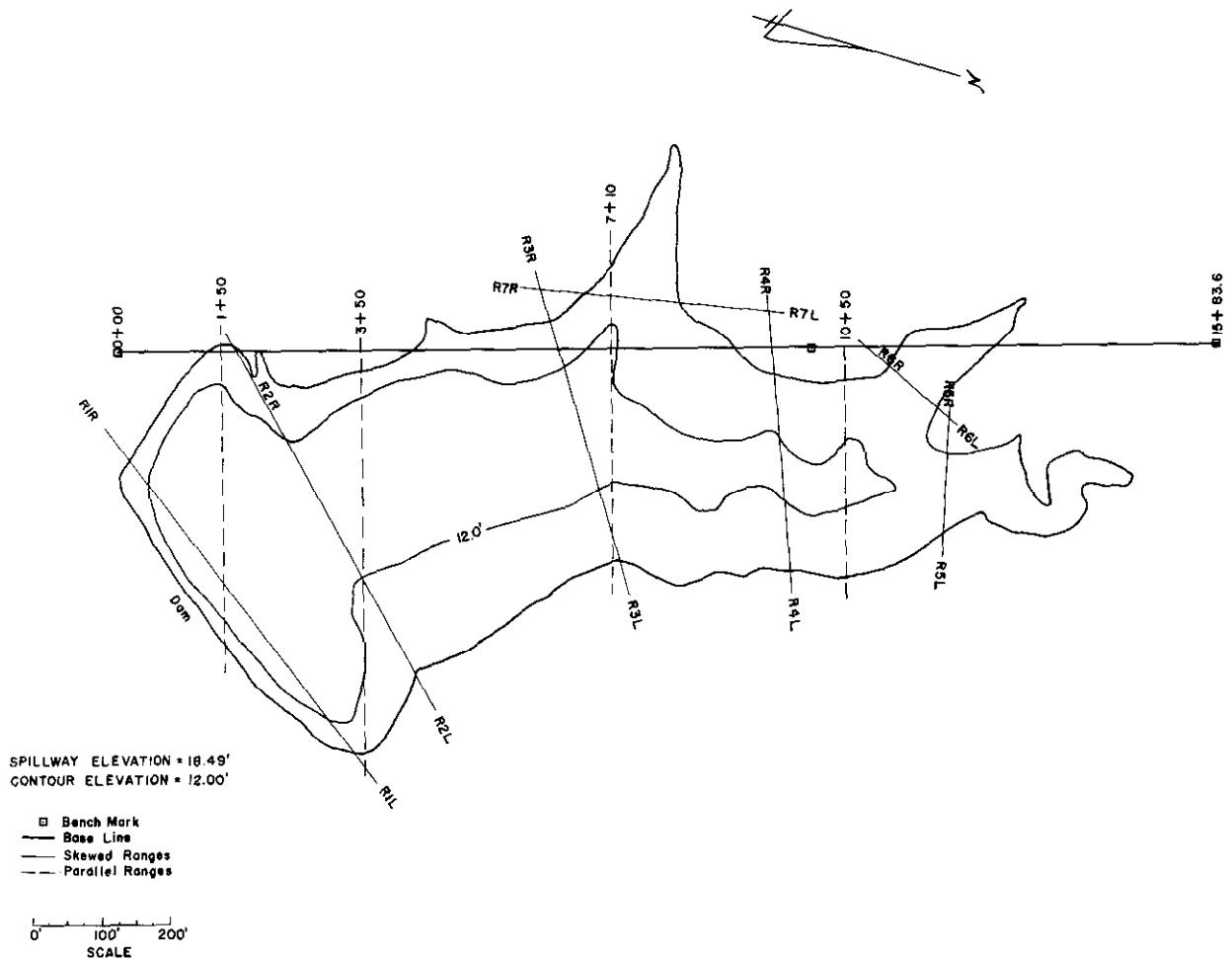


FIGURE 12. — Imposed survey, Al Olsen Reservoir.

tour areas or even spacing of ranges or contours.

Discussion and Conclusions

Volumetric Survey

The suggested survey procedure discussed previously was developed after considerable fieldwork experience in determining capacities. These experiences have shown that a base line along the side saves time. It permits the plane table work to proceed with measured distances always available for verification. With such a base line and ranges perpendicular to it, ranges can be located by distances along this line, and no range end monuments need be established.

The spillway and selected lower contours provide substantial information for drawing the other contours. The latter is helpful whatever its elevation, but the added information is minimized if this contour is relatively close to the

spillway contour or near the bottom of the reservoir.

There are instances where the base line along the side and determining the location of the lower contour are not practical. In these cases, select the best base line available and locate the range ends on the plane table sheet by distance measurements and by triangulation. Where the location of a selected lower contour is not practical in the field, the ranges should be spaced closer so that the topography can be determined adequately.

It is important to acquire adequate data in the field. The location of an extra range or two and the obtaining of information along such ranges take little time. The field surveys should be oriented toward the capacity calculation method.

The results tabulated in the preceding tables show that the detailed surveys actually made of

the four reservoirs were not necessary. A capacity value that is practically the same as that computed from the detailed survey was obtained with considerably fewer ranges carefully located, the upper spillway contour, and a selected lower contour. The suggested type of survey requires considerably less field and office time, fewer permanent markers, and less funds.

Methods of Computation

Experience elsewhere and this particular study show that the stage-area curve method is the most direct, simple, accurate, and uniformly adaptable way to determine the capacity of a reservoir. This method provides more easily the remaining capacity, the stage-storage curve, and other information.

This method also exploits more fully the advantage that all contour area methods have over the cross-sectional area methods; namely, that the stage-area curve can be plotted easily for the entire reservoir or segments of it. When the work is done by segments, information is obtained in regard to capacity changes with increased distance from the dam.

Experience in the association of reservoir topography with corresponding stage-area curves can be used in determining what fieldwork is needed to provide adequate data for capacity determinations. As an example, if experience in particular topographic areas indicates that stage-area curves are likely to be straight lines, minimum field survey data will probably be adequate.

The modified prismoidal method requires more calculations than the stage-area curve method; yet, it is not as accurate if the stage-area curve is irregular. Furthermore, judgment is involved in the calculations and there is added opportunity for errors.

The Simpson's rule requires an odd number of equally spaced contours or ranges. If this cannot be obtained easily, it often necessitates the use of another method to compute the capacity of the extra segment to complete the total capacity determination. The formula provides a close approximation of the actual capacity, but its use is limited.

The average contour area method is simple, but an average of two contour areas does not always approach the correct value, especially if there is a large difference in the relative sizes of two adjoining areas. Additional calculations are also required, which increases the possibilities of errors.

The capacity determination by Eakin's range formula requires a large amount of calcu-

lations using approximation equations. The equations depend on segment shape and location. Different ones are required for (1) the first segment next to the dam; (2) triangular-shaped segments caused by side tributaries; (3) long, meandering segments; (4) relatively straight segments; and (5) end segments. There are several equations for some segments, and the most appropriate one must be selected on the basis of judgment. Furthermore, the location of ranges is more critical when used with Eakin's method than with the stage-area curve method. This observation is confirmed from range layouts shown in figures 3 and 12 and the tabulations shown in table 1. Supplemental information cannot be readily utilized in this method even though it is easily obtained.

The cross-sectional area versus the distance from the dam method requires that the ranges be located approximately perpendicular to the valley and parallel to each other. There also must be a sufficient number of ranges so that the curve is well defined.

The average end area method applied to range cross-sectional areas is simple, but it also has the same disadvantage as the average contour area method. An average of two cross-sectional areas does not always approach the real one, especially if there are appreciable differences in the areas. Furthermore, this method cannot provide stage-area or stage-capacity information.

Whenever the ranges are approximately perpendicular to the valley and parallel to each other, a plotting of the cross-sectional areas of ranges versus the distance of the ranges from the dam can reveal shortcomings in the range layout. If this plotting shows irregularities that are not adequately supported by plotted points, then additional ranges need to be established at those locations to provide the missing data. When sufficient points are available so that the curve is accurately defined, the range layout and the associated volumetric determination will be adequate.

Acknowledgments

Larry Reynolds, hydraulic engineer; Elmer Mandrup, engineering technician; and Charles Hruby, formerly an employe, Agricultural Research Service, did most of the detailed office work in the experimentations with the different methods of computation. Other fellow Agricultural Research Service workers aided substantially in the field surveys and office computations and drafting. A. L. Sharp, supervisory hydrologist, Agricultural Research Service, was very helpful with his review comments.

DETERMINATION OF RESERVOIR DEPOSITS BY RECONNAISSANCE METHODS

[Paper No. 84]

By GLENN H. CASTLE, *supervisory hydraulic engineer, U.S. Army Engineer District, Sacramento*

Synopsis

Sedimentation estimates for many large reservoirs, such as those in the Sierra Nevada Range in California, indicate filling rates that will not significantly affect their operation for 50 to 100 years, unless the sedimentation is concentrated to an unforeseen degree at some critical point within the reservoir where it might interfere with the behavior of special works such as diversion intakes, bridge crossings, or nearby roads. In such reservoirs, occasional resurveys may be required, mainly to detect gross errors in estimated sedimentation rates or unusual or unexpected concentration of sediment at critical points, rather than for detailed information on exact sedimentation rates and information on specific gravity and grain size of sediments. Experience indicates that occasional aerial and/or ground reconnaissance of such reservoirs may be adequate for these purposes and can be accomplished for only a few hundred dollars rather than the thousands of dollars needed for a standard hydrographic or land resurvey of the established range system. When a reconnaissance indicates sufficient sedimentation to justify a more complete survey, information obtained by such reconnaissance can usually be used to reduce the scope of the required survey sufficiently to more than recover the cost of the reconnaissance. More use of reconnaissance methods is recommended.

Determination of Reservoir Deposits by Reconnaissance Methods

Current knowledge of the amount, type, and location of sediment deposits in a reservoir may be of major importance where sediment rates are relatively high and available space is limited, but deposits may be of only minor interest if rates are relatively low and available space is adequately large. A study of estimated sediment rates and of the planned size and method of operation of the reservoir will usually indicate the probable severity of the sediment problems and the frequency of required resurveys.

Most major streams rising in the Sierra Nevada Range of California have large parts of their watershed above the winter snowline, and a major portion of their annual runoff occurs during the spring months as a result of melting snow. Snowmelt runoff rates are moderate and result in relatively low sediment production rates, with most of the transported material consisting of sand and heavy silt moving as bed-

load. Major rainstorms also occur on these streams and result in high flows of short duration and low volume. These short duration flows have higher sediment production and transport rates, both as bedload of sand and gravel and as suspended load of silt and clay. The combination of large streamflow volume having relatively low sediment rates and small streamflow volume having higher sediment rates is such that the combined total annual sediment production and transport rates are relatively low on most of these streams. Although predicted annual sediment rates are low, it has been considered desirable to install permanently monumented sediment ranges in many of these reservoirs in order to measure total sediment deposition during their long anticipated lives. Where such ranges have been installed, adequate information on progressive sedimentation can often be obtained by only infrequent resurvey of the entire range system if this information is supplemented by more frequent reconnaissance surveys.

In the Sierra Nevada reservoirs, which are characterized by steep side slopes above relatively flat bottoms, most of the sediment is relatively coarse and heavy and is deposited near the head of the reservoir in well-defined steep-faced deltas close to the old stream channel rather than uniformly distributed over the reservoir bed and sides, as usually occurs when the incoming sediment is fine or largely colloidal. The characteristic wide-season variation in pool level and consequent movement of the head of pool up and down the canyon causes these delta formations to occur at widely varying points along the stream, but does not cause deposition on the side slopes. Therefore, sediment surveys can usually be limited to the flat bottom areas, at least for reconnaissance purposes. The scope of the survey can sometimes be further limited to a comparatively few ranges, because examination of the operational record of the reservoir will often indicate that most of the sediment accumulation should be found within a particular range of elevations.

The typical wide fluctuation of reservoir level that occurs in these reservoirs, because most of the annual flow is concentrated during one season of the year, also makes it possible to carry out a large part of a reconnaissance survey above water, if the survey is properly timed. Under these conditions it is sometimes possible to determine by aerial observation and aerial photographs or by aerial photographs, particularly

in color, where and when appreciable sediment has been deposited. Such aerial observations are very inexpensive and may be entirely adequate when only general information is required. If more detailed information is required, these observations may also indicate where additional surveys are needed.

For reconnaissance of the part of the reservoir area that remains under water, use can be made of a spud bar that will measure sediment depths of 10 to 15 feet. Figure 1 shows the spud bar used by the Sacramento District Corps of Engineers. This bar, which is 10.5 feet long and 1 1/4 inches in diameter, was manufactured at a local machine shop and is basically the same as the bar designed and used by U.S. Soil Conservation Service. Spud bars are best adapted to water depths of less than 100 feet but have been successfully used in depths of over 200 feet. Special precautions must be taken when working in deep water to prevent loss of part or all of the sample while the bar is being pulled up. The survey should be made in calm weather

when there are no large waves to cause excessive surging. The spud bar should be raised slowly so as to wash the sample as little as possible. Coating the bar with heavy grease prior to each measurement has been found to aid in retaining the sample; by this means samples have been obtained in water over 200 feet deep. The small grooves on the outside of the tip and the cup in the bottom of the tip are very helpful in retaining samples of the material at the deepest point penetrated.

Pine Flat Reservoir on Kings River, Calif., has been selected as an example to show the use of various reconnaissance techniques. This reservoir is located at the base of the foothills about 30 miles east of Fresno. The reservoir is created by a concrete gravity-type dam 440 feet high, with a crest length of 1,820 feet. Flows are released through two tiers of 5- by 9-foot conduits or over a gated spillway. The lower tier of outlets is located just above the riverbed at elevation 570 feet, and the upper tier is located 140 feet higher. The reservoir

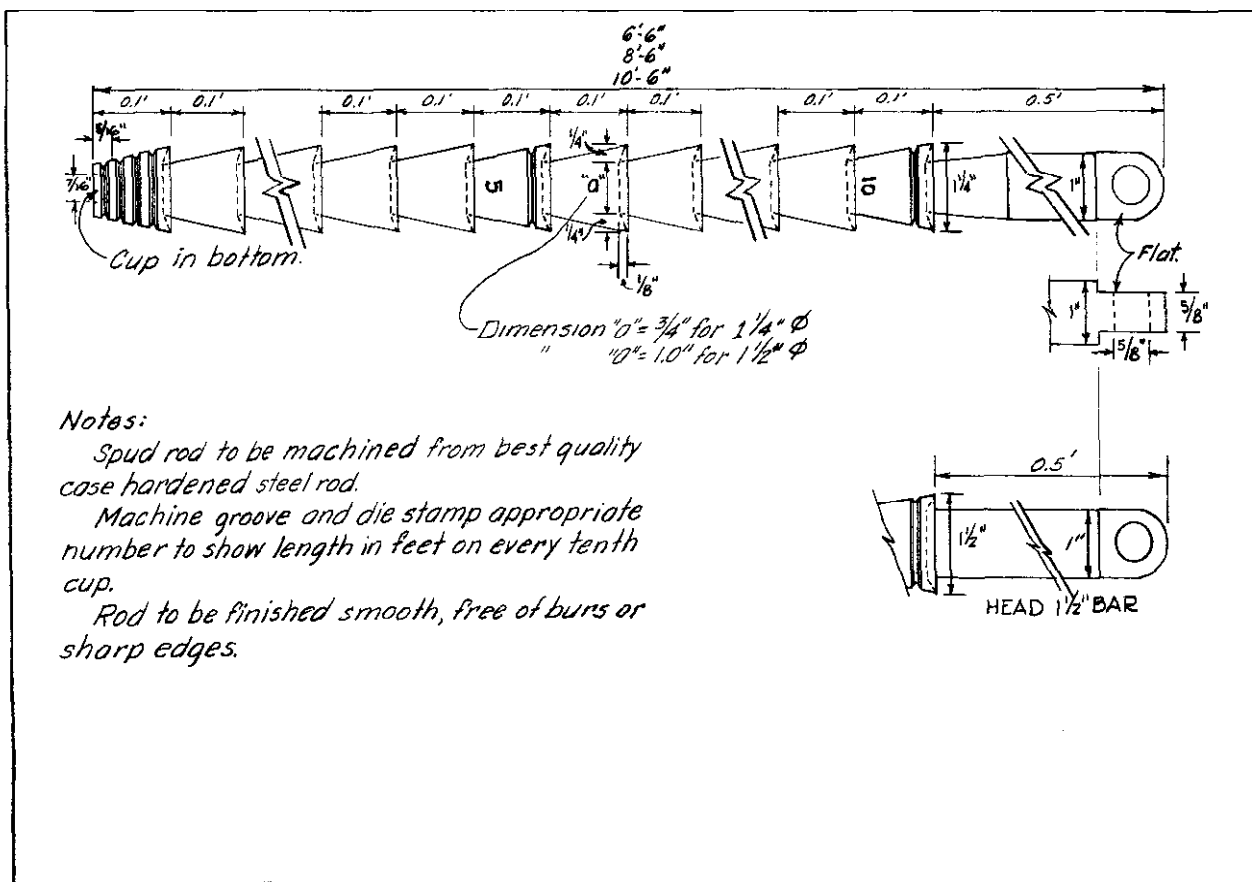


FIGURE 1. — Spud rod for sediment sampling.

was placed in full operation in 1954. Since that time, releases have been made only through the upper tier of outlets (or over the spillway since 1955), except for a short period in the summer of 1962. The reservoir at elevation of 951.5 feet m.s.l. contains 1 million acre-feet, covers an area of about 6,000 acres, and is 20 miles long. Reservoir operation is for flood control and conservation.

The drainage area above Pine Flat Reservoir covers 1,542 square miles and ranges in elevation from less than 1,000 feet to over 14,000 feet, with 65 percent of the drainage area above 6,000 feet. Mean annual runoff is about 1,650,000 acre-feet, of which about 1,190,000 acre-feet results from snowmelt. Inflow has ranged from 65 c.f.s. to 110,000 c.f.s. Annual rainfall ranges from about 15 inches in the foothills to over 40 inches at the higher elevations. Most of the precipitation falls as snow above about 6,000 feet. The higher mountain areas above 10,000 feet are characterized by little soil cover and large areas of exposed granite, whereas a large part of the area between 10,000 and 5,000 feet

has deep soils and dense forest cover. The area below about 5,000 feet is generally covered with brush and grass.

A study of probable sediment rates in various proposed reservoirs in the Sacramento District was made for the Corps of Engineers by the U.S. Soil Conservation Service in 1947. Results of this study, which was reported in U.S. Soil Conservation Service's Special Report No. 10, included an annual sedimentation estimate for Pine Flat Reservoir. Annual sediment inflow to Pine Flat Reservoir was estimated at 230 tons per square mile of drainage area. The trap efficiency of the reservoir was estimated at 96 percent and the specific weight of deposited sediment as 62 pounds per cubic foot. The annual deposition in the reservoir was therefore estimated at 220 tons or 0.16 acre-foot per square mile or about 250 acre-feet per year. This would amount to only about 12,000 acre-feet of deposition in 50 years, or slightly over 1 percent of the reservoir capacity.

Although sediment deposition in the reservoir did not appear to be a major problem that might

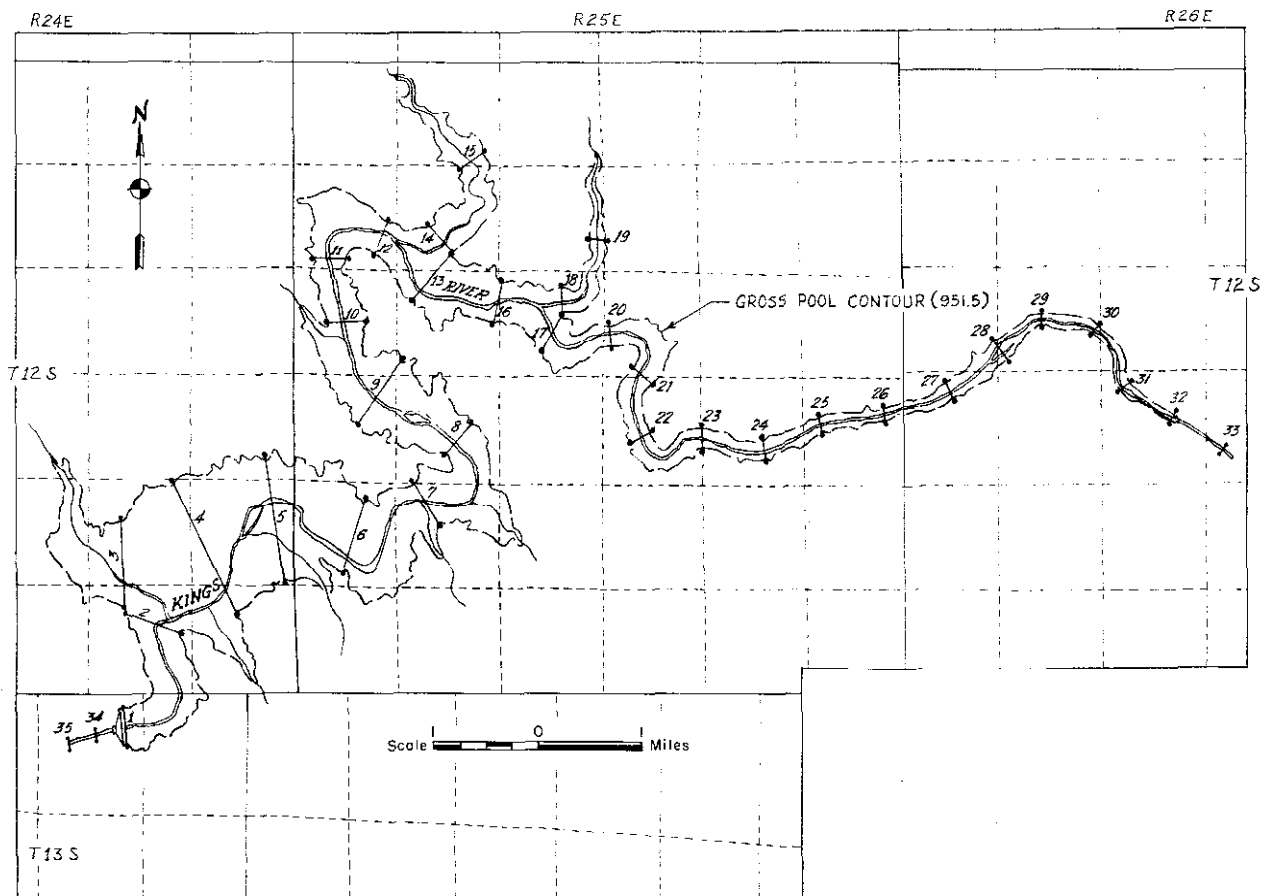


FIGURE 2. — Sediment ranges, Pine Flat Reservoir.

immediately affect reservoir operations, it was decided to establish a network of permanently monumented sediment ranges and to resurvey then at sufficiently frequent intervals to detect any gross errors that may have been made in estimates of quantity or location of deposits and as a means of measuring very long range sedimentation and changes in the reservoir capacity. This was highly desirable, since good sediment records showing both location and amount of sediment were not available for any comparable reservoir in the Sierra Nevada area. A system of 31 ranges was established in the reservoir, as shown on figure 2.

It also was decided to measure sediment outflow regularly, in order to verify the trap efficiency of the reservoir. This is accomplished by taking "grab" samples in the extremely turbulent area below the flip bucket. Comparison of several grab samples taken in this turbulent area indicate that each sample is representative and that integration of several samples is not required. Also, the same quality of results can be expected with grab samples under this condition as would be obtained with detailed samples of a normal river channel with the use of a standard sampler, such as a P-46. Samples are taken once each month, with special samples taken each time a sudden major change is made in the outflow, or if unusual turbidity occurs. Suspended sediment has varied from less than 1 part per million by weight to about 140 parts per million, with the latter value observed for only a short period during the 1955 flood when the reservoir was quite turbid. Normally, the reservoir outflow is clear, with a turbidity of

only 2 to 8 and suspended sediment of 1 to 3 parts per million. The calculated annual sediment outflow has been less than 5 acre-feet per year, thus indicating that the trap efficiency is probably in the order of 96 percent, as had been estimated by the U.S. Soil Conservation Service in their Special Report No. 10. This sampling program will be continued and reevaluation of the trap efficiency will be made at the time of the next complete resurvey of the reservoir.

While the estimated annual rates were believed reasonable, little was known of the amount of sedimentation that might occur during a single major rain flood. It was therefore decided to make a special sedimentation survey following the major rain flood of December 1955. Although a second rain flood occurred in January 1956 before the survey could be made, the reservoir stage during the January flood was about 150 feet higher than during the December flood. Since it was believed the sediment from each flood would be deposited mainly near the head of the current reservoir pool, it therefore appeared likely that the results of each flood would be largely separable, and if not, the total amount in the reservoir could still be obtained.

A reconnaissance survey was conducted in September 1956 while the reservoir was at its lowest stage since the occurrence of the December 1955 flood. Reservoir stages from beginning of storage in 1952 to October 1962 are shown on figure 3. Reservoir stage was at about elevation 740 at the time of the peak inflow in December 1955, and at elevation 875 at the time of the peak inflow of the January flood, and subsequently reached a maximum elevation of 941

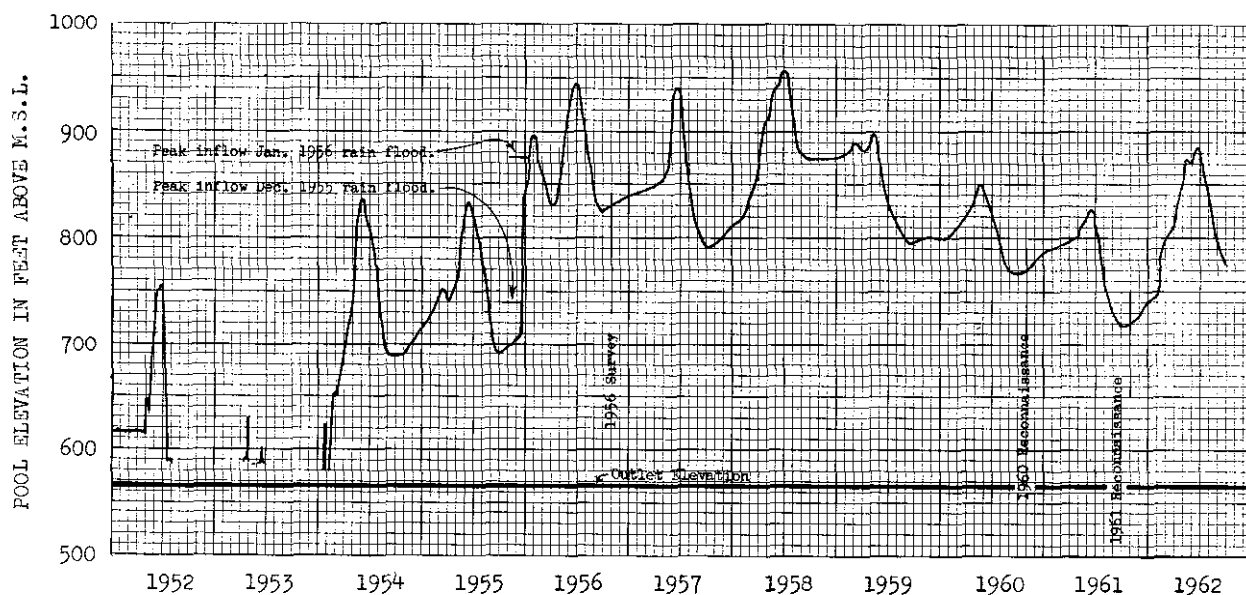


FIGURE 3. — Pine Flat Reservoir pool elevations.

feet during the spring of 1956. The reservoir had receded to elevation 826 in September 1956 when the reconnaissance survey was made.

All ranges above streambed elevation 826 were inspected and observations and estimates of deposition depths and amounts made. Depth of sediment was determined by digging with either a shovel or soil auger. Distances along the ranges were estimated from the reservoir topographic maps and the original detailed surveys of the sediment ranges. The portion of the reservoir area below elevation 826 was explored by use of the spud bar along the sediment ranges. Care was taken to sample at the deepest point along each range so as to be certain of obtaining samples in or near the old river channel. The bar was attached to 300 feet of copper core tiller line. The line was tagged at 10-foot intervals, and a large float was attached to the end so the line could not accidentally be lost when a sounding was made. The spud bar was lowered from a boat until bottom was found, then raised a few feet, and gently dropped to detect rocks or coarse gravel.

After locating a sampling spot free from such rocks and gravel, the bar was raised to the surface and dropped or plunged to obtain optimum penetration. The total depth from water surface to the point of the bar was measured by the tag line both before and after dropping for penetration. A simultaneous measurement of water depth at each point was made with a 3-pound bell sounding weight and standard tagged sounding line as an additional check on penetration depth. The differences in the two depths thus gave a check on depth of penetration of the spud bar. It was found that the bar would not penetrate more than about 1 foot of natural ground, but was capable of penetrating up to 10 or more feet in silt deposits. In nearly every instance where the spud bar penetrated to natural ground, samples of the material penetrated were retained even after withdrawal through over 100 feet of water. Coating the bar with grease aided in retaining the samples in deeper water. Therefore, even where deep water made it difficult to retain a sediment sample, it was usually possible to determine the approximate depth of sediment, either by examination of the retrieved bar sample, or by depth comparison.

Upon completion of the reconnaissance surveys, sediment depth at each sampling point and the position of each point on the cross section were plotted for all ranges surveyed and an estimate was made of the total sediment in the reservoir. This estimate indicated total stored sediment in the reservoir was probably between 1,300 and 2,100 acre-feet; thus, indicating a sedimentation rate of about 400 acre-feet per year. The average annual sediment rate had

been estimated at 250 acre-feet per year, but since this 4-year period of measurement included the very large flood of 1955, it seemed reasonable that the amount deposited would exceed the estimated average annual rate.

The reconnaissance survey indicated that sediment deposits along many of the sediment ranges were great enough to be effectively measured by soundings along the ranges. It was decided to make a detailed survey of every range in the reservoir except ranges 3 through 7, which were omitted since the reconnaissance indicated no measurable sediment in these ranges. The detailed survey utilized chained cross sections above the pool level and lead line soundings, by means of boat and tag line within the pool. Lines of equal sediment depths were plotted as shown on figure 4 and the volume of sediment was computed by planimetry of the areas so defined. The total sediment volume was thus calculated to be 1,450 acre-feet, which was within the range of the reconnaissance estimate. The deepest deposits were found near range 16, which crosses the reservoir bottom at elevation 730, just below where the tip of the reservoir was at the peak inflow during the December 1955 flood. Figure 4 shows the heavy deposits between range 13 and ranges 17 and 18. Of the total 1,450 acre-feet of sediment, about 420 acre-feet, or 30 percent, was found between ranges 12 and 17 and apparently was mostly deposited during this large flood.

From 1956 to 1960 reservoir stages remained relatively high and no resurveys of any kind were attempted. However, in December 1960, reservoir drawdown reduced the stage to about 773 feet, 50 feet below the level at the time of the 1956 survey. A ground reconnaissance was made and it was observed that there had apparently been considerable downstream movement of sediment from the upper ranges and most of the sediment above the tip of the reservoir (at range 17) had moved downstream and formed a very long delta along the river bottom. This delta varied in width from about 400 feet on the downstream end to less than 100 feet at the upstream end and appeared to be somewhat unstable and continually progressing downstream as the reservoir receded. Flows were distributed evenly over this delta and had not formed narrow cuts in it as might be expected. This was believed due to the fact that reservoir inflow had been very low during the drawdown period, and it was felt that such channels would be cut and the silt and sand carried further down the reservoir when flows increased. Observations during the winter of 1960 showed this to be true, and considerable sediment movement was noted as normal winter and spring flows came into the reservoir.

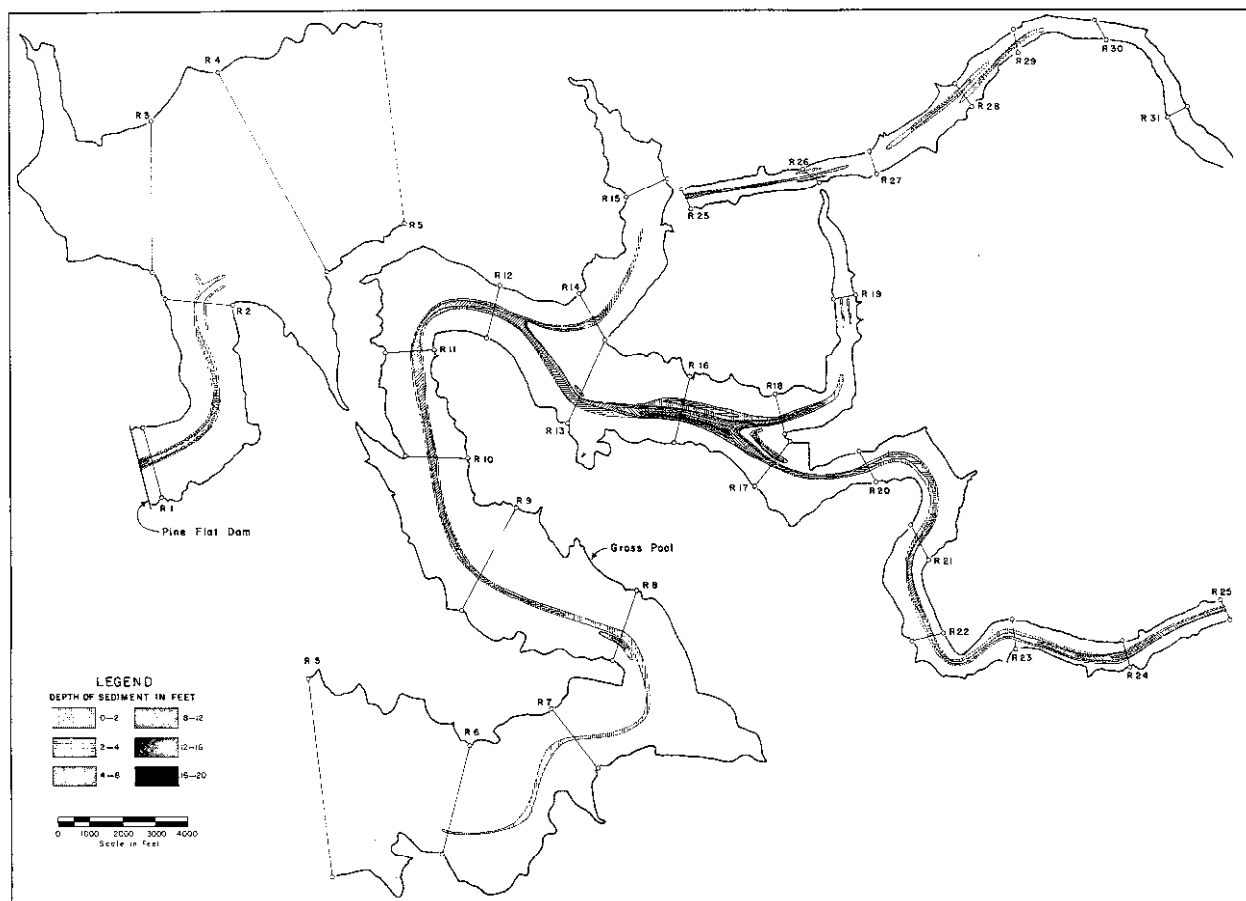


FIGURE 4. — Sediment deposition in Pine Flat Reservoir, May 1952 to November 1956.

During the summer of 1961 the reservoir was drawn down to elevation 718, the lowest stage since the summer of 1953 and 20 feet below the pool level that existed at the peak of the 1955 flood. This provided an excellent opportunity to resurvey the reservoir area above the pool level by use of reconnaissance methods. Aerial observations and photographs indicated that, as in 1956, most of the sedimentation was on the relatively flat reservoir bottom, with only very minor amounts on the steeper hillsides. Figures 5 and 6 are aerial photographs of the reservoir areas in the vicinity of ranges 16, 17, and 22, with the range lines superimposed on the photographs. Stumps, rocks, and gravel can be seen on the hillsides and on the outside of the river bends, thus indicating no deposition. Sandbar formations can be seen in the old channel and sediment of up to 10 feet deep was subsequently measured on range 17. The aerial observations also indicated that in certain reaches of the reservoir more deposition had occurred between ranges than along the ranges.

In planning the range layout, more consideration was given to locating the ranges where they would be representative of sediment deposits after a long period of reservoir operation, such as 50 years, than during the first few years of operation. It was felt that ranges so located would also be reasonably representative of depths between ranges during the accumulating period and that sediment volume could initially be computed by methods other than the end-section method if that method appeared unsuitable.

For economic reasons consideration was given to placing range monuments at points where horizontal control points had already been established during the topographic survey of the reservoir. These points were often on "noses" and as a result the range lines between these points generally cross the reservoir at sections having somewhat less than the average width. Consequently, there are more off-channel flat areas in the reservoir bottom between the ranges than are represented by the ranges. Sediment



FIGURE 5. — Sediment deposits, ranges 16, 17, 18, Pine Flat Reservoir.

deposited in these off-channel flats or terraces is likely to be "benched" and not moved and redeposited during times when high flows occur and the reservoir is at stages below their level. Such was the case at Pine Flat and as a result, some sections of the reservoir have more deposition than had been represented by surveys along the existing ranges.

Preliminary ground reconnaissance of the entire reservoir above the water level confirmed the aerial observations, and a reconnaissance type resurvey of all ranges above the reservoir was made. As in 1956, depths were obtained by digging through the sediment with a soil auger or shovel and by observing depths of vertical cut banks where possible. Position of the various points on each cross section was determined by estimating distance from recognizable points on the cross sections and the reservoir topographic map. Many points between the ranges were also examined and sediment depths plotted on the reservoir topographic map. Lines of equal sediment depth were then plotted on the

reservoir map as shown on figure 7 and planimeted as after the 1956 survey.

It was assumed that only minor deposition had occurred below range 11 between 1956 and 1961, since the reservoir had been considerably above that level during the entire period prior to the 1961 drawdown. The total sediment deposit so measured was about 1,950 acre-feet, indicating additional deposition of about 500 acre-feet since the complete survey of 1956. This deposit of only 500 acre-feet in 5 years or about 100 acre-feet per year is less than half the predicted annual amount. However, the total of 1,950 acre-feet in the 9 years of reservoir operation shows an average annual sediment deposition of about 220 acre-feet, very near the original prediction of 250 acre-feet.

Figures 8 and 9 are reservoir profiles which show maximum observed sediment depths on the various cross sections during the 1956 and 1961 surveys, respectively. Comparison of these profiles shows that most of the sediment accumulated since 1956 is above range 17, but below

range 23. It can also be noted that at least a part of this sediment was probably eroded from the area between ranges 23 and 27, where it was deposited prior to 1956. Examination of figures 4 and 7 also show that erosion of sediment occurred above range 23 and additional deposition occurred below range 23.

Although the main advantage of a reconnaissance survey is lower cost, poorer results might be expected, but this may not always be true.

Inspection of figures 4 and 7, particularly in the vicinity of ranges 13, 14, and 15, indicates that a more accurate picture may sometimes be obtained by a reconnaissance survey than from



FIGURE 6. — Sediment deposits, range 22, Pine Flat Reservoir.

a range survey, due to the examination of areas between the ranges. It is realized that where sediment deposits are of great depth and are uniformly distributed this would not be the case. Only 6 man-days were required in the field for the 1961 reconnaissance survey and about the same amount of time was spent in

office analysis. Thus, the reconnaissance survey was accomplished for about \$500 as opposed to the estimated cost of \$10,000 for a detailed re-survey of all ranges, using standard survey procedures. In the case of Pine Flat Reservoir, it appears highly improbable that the 1961 estimate could be as much as 50 percent in error

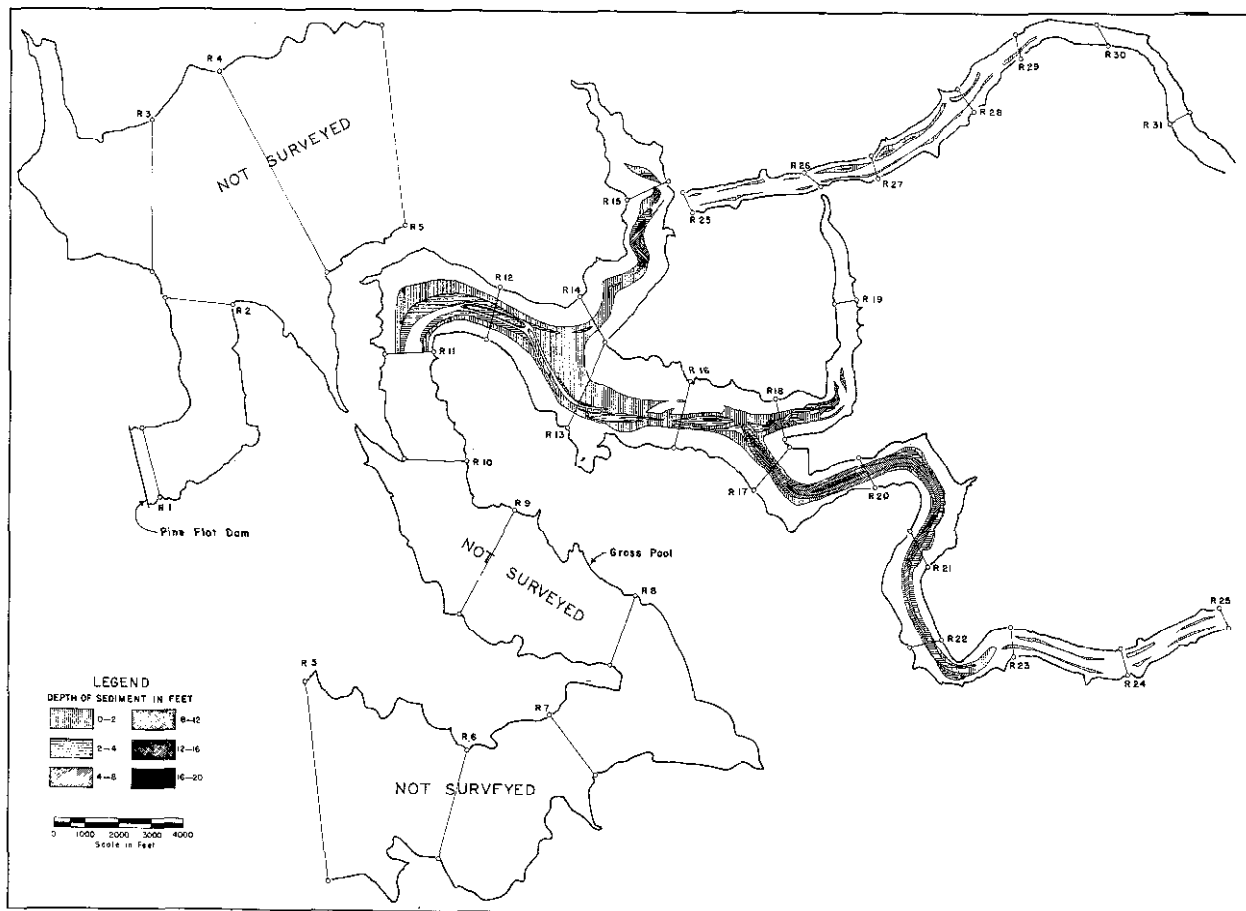


FIGURE 7. — Sediment deposition in Pine Flat Reservoir, May 1952 to November 1961.

and the results are considered adequate for project requirements.

From these studies of sedimentation in Pine Flat Reservoir, it is concluded that sufficiently accurate results may often be obtained by reconnaissance surveys in reservoirs where (1) rates are relatively low, (2) total capacity is large, and (3) current information is required to determine whether inlet or other project works are likely to be affected by sedimentation. Many reservoirs in the Sierra Nevada Ranges in California probably are in this category. It is believed other reservoirs in other areas of the

country will have these same general characteristics. In such cases it appears that reconnaissance surveys can be used as an economical method of making interim surveys to supplement more detailed surveys made at infrequent intervals. It is further concluded from these reconnaissance surveys that, in reservoirs of this type, care must be taken in planning a range layout or data obtained from the ranges alone may not be representative. Also, reconnaissance surveys should be used as a method of checking the adequacy of a range system early in the life of the project.

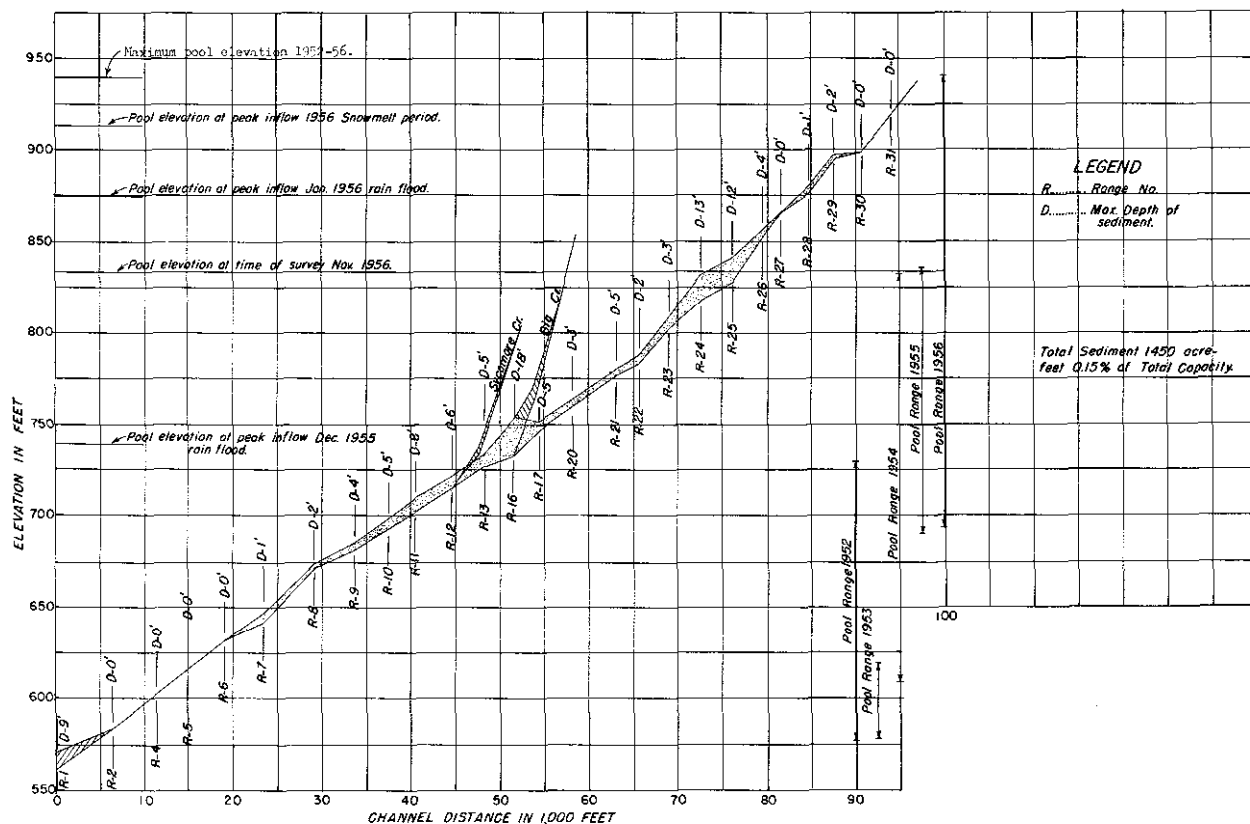


FIGURE 8. — Distribution of sediment in Pine Flat Reservoir, November 1956.

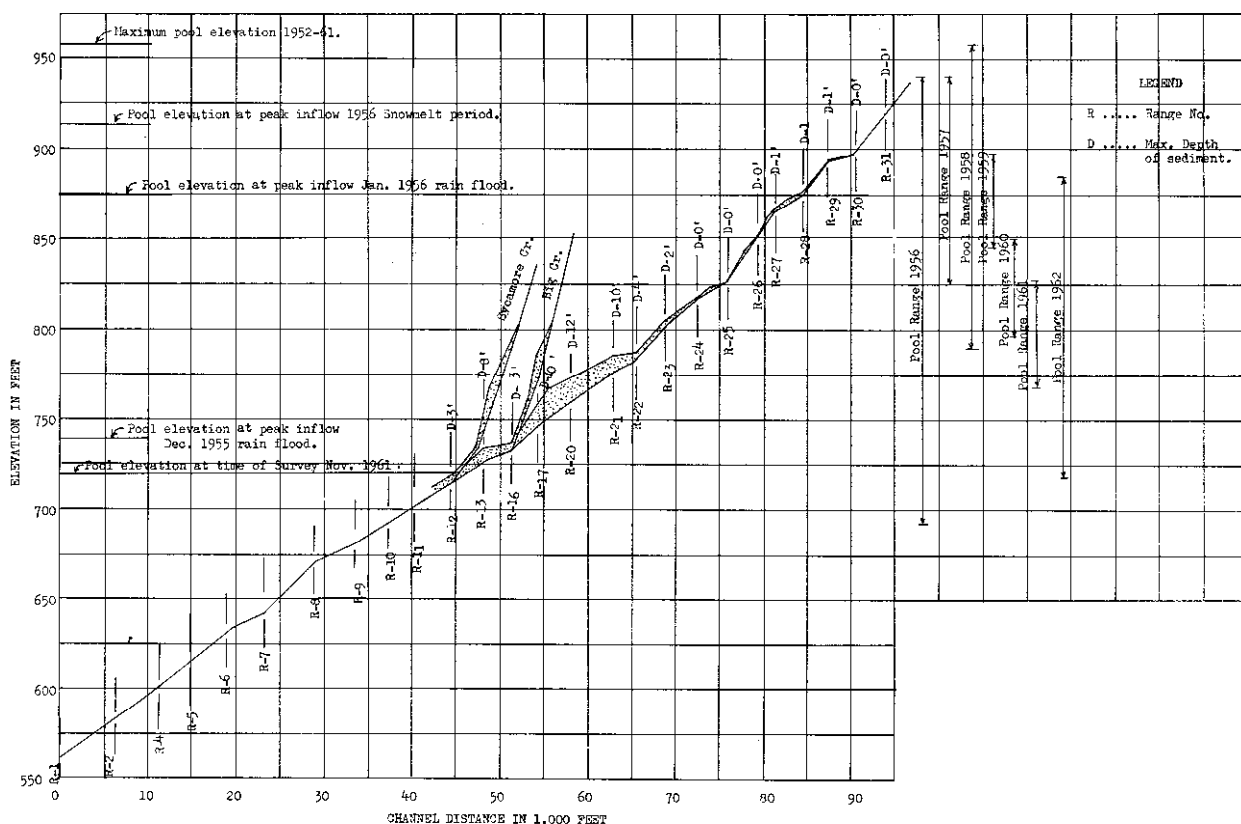


FIGURE 9. — Distribution of sediment in Pine Flat Reservoir, November 1961.

RECONNAISSANCE-TYPE RESERVOIR SEDIMENTATION SURVEYS

[Paper No. 85]

By ALBERT L. HILL, *hydraulic engineer, U.S. Army Engineer District, Kansas City*

Synopsis

Reconnaissance-type reservoir sediment surveys can be made at a reasonable cost and with sufficient accuracy to meet many operational and administrative needs. This technique should become more effective as additional experience is gained with reservoirs of various sizes and with the types of equipment now available. Three reconnaissance-type surveys were made in the Kanopolis Reservoir on the Smoky Hill River in Kansas. Point soundings were made on selected ranges and the results plotted on the original cross sections. The sections were completed by extrapolation and the sediment volume computed. Results of these three surveys were found to be from 1 to 10 percent higher than the estimates from sediment inflow-outflow data. A complete survey made in 1960 showed the sediment volume to be 14 percent higher than

that estimated from the inflow records. This kind of agreement indicates an acceptable degree of accuracy for the reconnaissance-type survey.

Introduction

The larger reservoir projects and many small and medium projects have sediment-observation programs that include inflow and outflow sampling stations and a system of sediment-observation ranges, or a substantial part of these provisions. A complete resurvey of sediment ranges may cost from \$0.25 to \$0.50 per acre, depending on amount of exposed soft sediment, spacing of ranges, and the extent of weeds, woody growth, or field crops that may require clearing a part of some ranges. An abbreviated-type survey may consist of the "armchair" survey, computing inflow and outflow and estimating the location of deposits from reservoir-stage

records, or the reconnaissance-type surveys consisting of (1) checking exposed sediment depths and submerged deposits' thickness by probing, (2) visual inspection of exposed deposits and cruising the permanent pool with an echo-sounder that will reflect sediment thickness, (3) complete survey of a few selected ranges, and (4) a number of point observations of sediment depths in the permanent pool and exposed deposits.

The principal reasons for making the abbreviated- or reconnaissance-type survey are (1) low cost, (2) evaluating the need for a complete survey, (3) for administrative purposes at a multiple-purpose project where different storage allocations are involved, (4) measuring sediment deposits at boat harbors and launching facilities, and (5) investigating mud flats or swamp areas.

Three of the reconnaissance-type surveys were made in the Kanopolis Reservoir, situated on the Smoky Hill River in central Kansas, as interim observations. The Kanopolis Reservoir was constructed by the U.S. Army Engineer District, Kansas City, and has been in operation since 1946, primarily for flood control. The total initial storage capacity was 450,000 acre-feet, of which 53,000 acre-feet was for conservation and sediment requirements. At full-pool elevation of 1,508 feet m.s.l., the reservoir has an area of 13,900 acres and is 25 miles long. The conservation-sediment pool, at elevation 1,459 feet m.s.l., has an area of 3,550 acres and is 12 miles long. Prior to filling the reservoir, a series of 27 sediment ranges were established and surveyed across the main reservoir and 10 ranges were established and surveyed across tributary arms.

Suspended Sediment

Prior to construction of the dam, a sediment-sampling station was established at the stream-gaging station at Ellsworth, Kans., located immediately upstream from the reservoir on the Smoky Hill River. Sampling the outflow from the dam was started soon after impoundment began. These data were used to estimate inflow and outflow tonnage and, together with particle-size data, to compute volume of sediment deposits with due regard to age and degree of submergence.

Surveys

Fieldwork consisted of taking soundings at a number of points on selected ranges in the permanent pool and determining the thickness of deposits at selected points on that part of the

ranges above the permanent pool. The monuments were located and flagged, and transit and stadia used to locate observation points on each range examined.

Office Work

Office work consisted of plotting field observations on the range profile and extrapolating to complete the section. The areas were planimeted and the volume of sediment computed by the end-area method. Results of the survey were compared with estimates prepared from inflow and outflow data. For the purpose intended, no further reports were necessary.

Results of Surveys

In 1960 a complete survey was made of the reservoir¹ and findings of the survey compared with estimates made from inflow-outflow data. Table 1 shows the results of the reconnaissance-

TABLE 1.—Comparison of estimates from inflow-outflow data with survey results, 1960

Date	Survey type	Estimated volume of sediment in Kanopolis Reservoir		
		Inflow-outflow data	Survey data	Inflow-outflow
		<i>Acre-feet</i>	<i>Acre-feet</i>	<i>Percent</i>
Nov. 1950	Reconnaissance	4,140	4,300	+104.0
Nov. 1951	Reconnaissance	7,100	7,180	+100.4
Sept. 1954	Reconnaissance	7,230	7,990	+109.5
Sept. 1960	Complete	12,370	14,380	+114.0

type and complete survey as compared with estimates prepared from inflow-outflow data. In addition to volume comparisons, table 2

TABLE 2.—Approximate percentage distribution of Kanopolis Reservoir sediment (by volume)

Location	November		September	
	1950	1951	1954	1960
	<i>Percent</i>	<i>Percent</i>	<i>Percent</i>	<i>Percent</i>
In channel in conservation pool.	43	33	35	25
In channel upstream of conservation pool.	2	4	10	8
In overbank in conservation pool.	40	36	31	50
In floor of flood-control pool	15	27	24	17

shows location determinations from the surveys. Results of the reconnaissance-type surveys agree very well with inflow-outflow data. Although the degree of accuracy of either is not readily apparent, taken together, they add up to reasonably convincing evidence that the results are reliable.

Discussion

Results of this nature may be obtained by expenditure of a few hundred dollars for reconnaissance as compared to several thousand

¹ Results are contained in report "Sedimentation in Kanopolis Reservoir, Smoky Hill River, Kansas," prepared by U.S. Army Engineer District, Kansas City, dated December 1961.

dollars in the case of a complete survey on reservoirs comparable in size to the Kanopolis project. Improved equipment and methods now available should give even better results where the range method is used. For example, the U.S. Army Engineer District, New Orleans, has developed a piano-wire distance-measuring device that is valuable in measuring distances along the submerged part of the range. Annual maintenance of monument markers, so that they are readily located, helps to save time. In multiple-purpose projects, with on-line range monuments set near the water level of the permanent pool, a piano-wire distance-measuring device, and echo sounding equipment, a high-type reconnaissance survey is now possible. A complete reservoir survey involves a sizable sum of money. With a limited budget and the rapidly increasing number of reservoirs, it is necessary to resurvey only when the volume of sediment

reaches the point where it affects reservoir operation or necessitates reallocation to the various uses.

Conclusions

The Federal Inter-Agency Sedimentation Committee has performed a very valuable service in collecting sediment data and distributing it to the various using agencies. In this connection, the collection of data sheets for all known sedimentation surveys is of great value. As the volume of sedimentation data increases and the individual investigator becomes more experienced, a fairly accurate picture of sediment accumulation in a reservoir may be constructed, based on the rather limited observations of a reconnaissance-type survey. Ingenuity in planning limited observations and sound judgment in extrapolating the data are the primary requirements for using the reconnaissance-type reservoir survey.

USING RAYDIST FOR SEDIMENTATION SURVEYS ON LARGER RESERVOIRS

[Paper No. 86]

By ISAAC SHEPHERDSON, *hydraulic engineer, U.S. Army Engineer District, Omaha*

Synopsis

The Garrison District of the Corps of Engineers purchased a single dimension type ER Raydist system in 1956 for the purpose of resurveying the many long sedimentation ranges on Garrison Reservoir. Raydist accomplishes distance measurement by the phase comparison of separate radio waves from two continuous wave transmitters, one that must be stationary and the other that is mobile. This system requires the simultaneous operation of three transmitting-receiving units, which consist of a master station, a shore station, and a mobile transmitter. The usual arrangement is to place the shore station at one end of the range line and operate the master and mobile stations from the sounding launch to record distances from the known shore station. Since only the distance from a known point is established, the centerline guidance of the launch is maintained by transit through radio voice communication.

Advantages of this equipment are (1) convenient horizontal control for sounding long ranges; (2) reduction in visibility problems; (3) continuous position determinations; (4) ability to reposition very closely; (5) increased accuracy; and (6) simplification of office computations. Disadvantages and limitations dur-

ing calibration and operation of the equipment are (1) inaccuracies when the master station is located within 600 feet of the shore station or is near steep bluffs; (2) necessity for a restart when any unit fails during a sounding run; and (3) the need for a trained electronic technician to install and maintain the equipment.

The initial expense of this Raydist system necessitates a practical justification for use in sedimentation survey work. This requirement can be satisfied for such large reservoirs as Garrison, where some range lengths exceed 4 miles. However, for the normal operation on smaller reservoirs other methods of horizontal control, such as stadia, transit intersection, distance measurement by piano wire, or towing a current meter, must be considered as the more appropriate technique.

Introduction

Raydist was mentioned in an article published in the proceedings to the 1947 Inter-Agency Sedimentation Conference held in Denver, Colo.¹ This article led to an investigation of Raydist by the Omaha District as a means of measuring distance for surveying the sedimentation ranges on Garrison Reservoir.

At maximum pool elevation, approximately 72 of the ranges on Garrison Reservoir will have underwater lengths between 1 and 7 miles. Because of the large number of long ranges, sev-

¹ Discussion by Charles W. Thomas to article titled "Reservoir Sedimentation Surveys—Objectives and Methods," by Albert S. Fry. 1947 Fed. Inter-Agency Sedimentation Conf. Proc., Denver, Colo. 1948.

eral methods of horizontal control for resurveying these ranges were investigated.

The *stadia and distance measurement by piano wire* methods were satisfactory for ranges approximately 4,000 feet in length.

Towing a current meter² had been used for surveying long ranges; however, this method had disadvantages because of (1) time and computations necessary to prorate the error accumulated for a particular survey; (2) errors due to the effect of wind and waves on current meter readings; and (3) possible reruns when current meter distance differ substantially from the actual distance.

Transit and plane table intersection was considered adaptable for the survey of long ranges; however, there were disadvantages because of (1) necessary surveys and cost for establishing points for using this method; (2) problems due to visibility because of fog, haze, and in some instances timber; (3) the necessary computations (in the case of using the transit) for determining the stationing for the fix lines on the sounding chart; (4) reduced accuracy (in the case of the plane table) in transferring the stationing from the plane table to the fix lines on the sounding chart.

Raydist appeared to have several advantages over other methods for the sounding of long ranges: (1) convenient horizontal control for sounding long ranges; (2) reduction in visibility problems; (3) continuous position determinations, i.e., personnel in the sounding launch would know their position at all times; (4) ability to reposition very closely, i.e., resounding of parts of a range could be easily accomplished and following sounding of a range it would be easy to return to sampling locations selected from the sounding chart; (5) simplification of office computations; and (6) elimination of the cost of establishing points for cutting in the boat by transit or plane table.

No disadvantages in the use of Raydist for horizontal control on sedimentation surveys were discovered upon investigating its use by the Norfolk District, and the U.S. Lake Survey Corps of Engineers.^{3 4} Although the Lake Survey group did dispose of its type E Raydist system, its reasons for disposal did not apply to

the work the Omaha district planned for the equipment. The purchase of Raydist was justified solely by the sedimentation surveys planned for Garrison Reservoir; however, it was envisioned that the Raydist system might be used on other reservoirs and by other Districts. Over one-half the initial cost of Raydist was justified by the elimination of the cost of establishing points for the use of the transit and plane table methods.

Consequently, in 1956 a single dimension range part of a type ER Raydist system was purchased from Hastings-Raydist, Inc. The equipment purchased was such that the additional components could be added to make a complete type ER Raydist system. Thus far, the Omaha district has not needed a two-dimensional system, because detailed hydrographic surveys of extensive areas have not been required. The district also contracted for a week's work by one of the Raydist technicians to help in installing, setting up, operating, and testing the equipment.

Briefly, Raydist accomplishes distance measurement by the phase comparison of the wavelengths from two continuous wave (CW) transmitters, one of which is moving and the other is stationary. The equipment literally counts the radio waves through which it moves. A single dimension Raydist system requires the simultaneous operation of three transmitters and three receivers and is made up of three distinct groups of equipment: the master station, shore station, and mobile (CW) transmitter (fig. 1). The phase meter in the master sta-

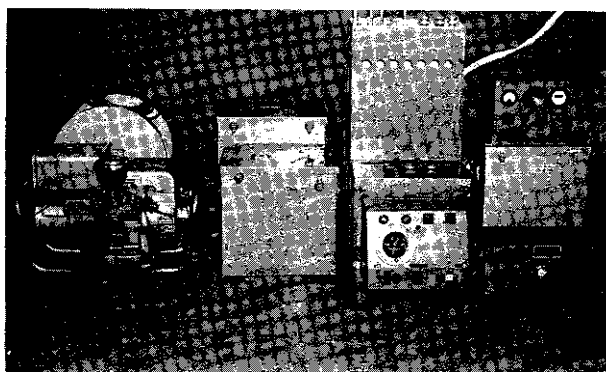


FIGURE 1.—Equipment comprising the three Raydist groups: Left to right are (1) generator and cable, (2) shore station units, (3) master station group, and (4) the test set, mobile transmitter and master power supply.

tion measures distance in lanes. A lane is equal to one-half the wavelength of the mobile (CW) transmitter.

² Report of sedimentation survey, Denison Dam and Reservoir (Lake Texoma) by the Tulsa District, Corps of Engineers, June 1950.

³ "Raydist for Shallow-water Hydrography," by Harold H. Waterfield, engineer, chief, Survey Branch, Norfolk District, and Cecil Hilliard, engineer, in the Norfolk District, November-December 1951, Military Engineer.

⁴ "Raydist Preliminary Report," dated January 1, 1952, and "Final Report Test of the Type E Radist for Hydrographic Charting Over Fresh Water," dated June 30, 1953, U.S. Lake Survey, Corps of Engineers.

Arrangement of Raydist Units

The three basic groups of Raydist equipment are the master station, the shore station, and the mobile transmitter. These can be arranged to give three different position configurations: circular, elliptical, and hyperbolic.

The circular arrangement is achieved by operating with the mobile transmitter integral with the master station, i.e., it moves with the master station or remains fixed with the master station. In this arrangement either the master station or the shore station may be fixed or mobile and the phase meter at the master station will continually indicate the circular line of position upon which the mobile equipment is located. Lines of position are a family of concentric circles centered upon the fixed equipment. (At either group of equipment the measuring point, center of radiation, lies halfway between the CW transmitter antenna and the AM receiver antenna.) Figure 2 shows this arrangement and the resulting position configuration.

The elliptical arrangement is achieved by separating the mobile transmitter from the master station (forming three groups) and

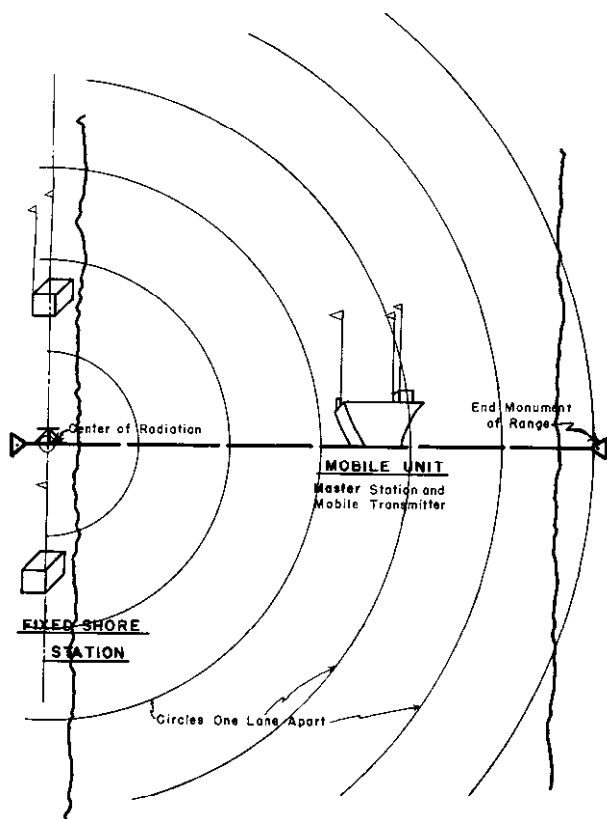


FIGURE 2. — Circular arrangement and resulting configuration pattern.

making the shore station mobile while the master station and mobile transmitter are fixed. The position configuration is a family of ellipses having their foci at the antennae of the mobile CW transmitter and the AM receiver in the master station. The phase meter will now indicate the ellipse line-of-position upon which the mobile unit is located. The ellipses are spaced a half-wave apart on the baseline extensions.

The hyperbolic arrangement is also achieved by using three groups of equipment. The mobile transmitter is used as the mobile unit, while the master station and the shore station are fixed. The indicator will now identify the position of the mobile unit on a family of hyperbolic lines of position having as their foci the AM receiver antennae at both fixed stations. The lines of position are spaced a half-wave apart along the baseline and gradually diverge with distance away from the baseline. In this case the baseline extensions also constitute lines of position (or lines of constant phase difference), and maximum and minimum phase meter readings will be obtained when the mobile transmitter travels (by any path) from any point on one extension to any point on the other extension. The difference of these readings will indicate the length of the baseline in lanes or $\frac{1}{2} f$. Figure 3 shows this arrangement and the resulting lines of position.

Pure range or true distance is measured along the line connecting the two or three groups of equipment comprising the circular and hyperbolic arrangements. The circular arrangement with the master station moving is the one most used, because position is shown at the master station for the mobile transmitter, which is a part of the master station in this case. The arrangement for the hyperbolic configuration has been used for control on long, shallow ranges, i.e., ranges that have areas too shallow for the draft of the large sounding launch.

Operating Principle of Raydist

The type ER Raydist system, as do all Raydist systems, employs the method of phase comparison of continuous wave (CW) signals. In the Raydist method, however, the phase comparison takes place at a radio frequency, thereby eliminating the problems of phase comparison at radio frequencies while maintaining the sensitivity and accuracy obtainable at the higher frequencies. This is accomplished by the use of two CW unmodulated transmitters operating on essentially the same frequency but slightly displaced by an audio amount. For example, one transmitter is operated at a frequency f and the second transmitter is operated at a frequency $f + \Delta f$, where Δf is an audio frequency

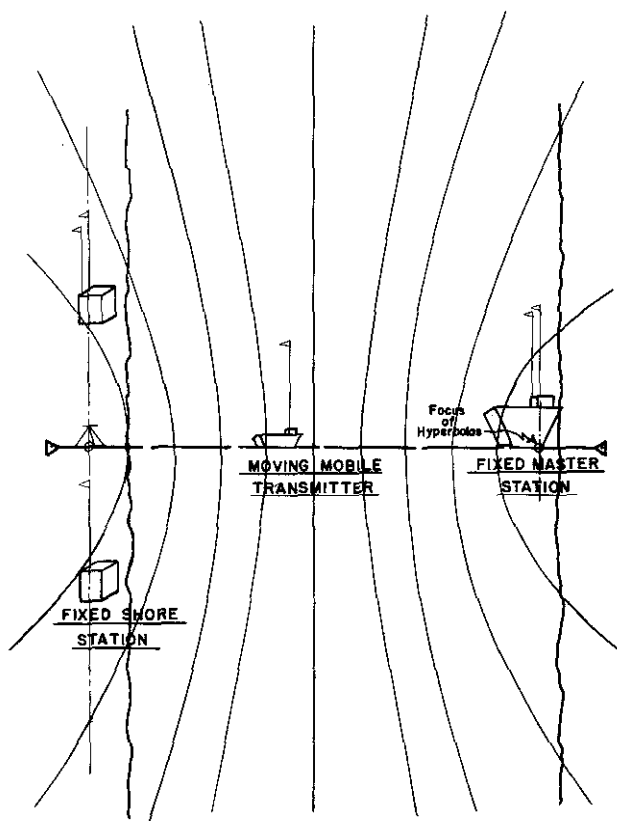


FIGURE 3. — Hyperbolic arrangement and resulting configuration pattern.

of usually 400 c.p.s. Receivers at the master station and shore station receive both carriers and detect a heterodyne or beat frequency, Δf . This heterodyne tone, as it is received at the shore station is returned to the master station and compared in the phase-measuring circuits, to indicate position. Since the heterodyne is derived from a fixed transmitter and a moving transmitter, its phase will be dependent upon the location of the mobile transmitter. Since both the master station and the shore station contain an AM receiver that must receive both of the CW transmitters and since both also contain those transmitters, it is obvious that each transmitter must be made to appear weak to the adjacent receiver but at the same time strong to distant receivers.

Special measures are required to permit these receivers to receive the strong local carrier and the relatively weak distant carrier simultaneously. This is accomplished by the use of nearly harmonically related frequencies and by making use of the second harmonic radiation at the shore station. For example, assume the primary basic frequency of the system to be f . This,

then, is the frequency that determines the lane width of the system, i.e., the spacing of the lines of position. The mobile (CW) transmitter at the master station is operated on the frequency f and the (CW) transmitter at the shore station is operated on $\frac{f \pm \Delta f}{2}$. The AM receiver at the master station is a dual channel receiver that receives the local carrier f on one channel, and the distant carrier $\frac{f \pm \Delta f}{2}$ on the second channel. These two signals are combined in the receiver to produce a single heterodyne audio output of Δf . The AM receiver at the shore station is tuned to f and receives the distant carrier from the mobile (CW) transmitter as well as a small amount of the local transmitter's

second harmonic radiation on $\frac{f \pm \Delta f}{2}$. The two signals heterodyne in the receiver to produce the audio beat of Δf . The audio tone, Δf , as detected at the shore station is then returned to the master station, via FM radio, for phase comparison. The results of the phase comparison are shown in lanes on the dial of the remote phase indicator. Figure 4 is a diagram of a two dimensional type ER Raydist system.

Radio frequencies allocated for the operation of this Raydist system are:

	Kc
Mobile CW transmitter.....	4,796.4
Shore station CW transmitter.....	2,398.0
FM transmitter	36.8

Transportation of Raydist Units

Shore Station

The shore station Raydist units and the necessary equipment to support the work required in setting up and operating the station are generally transported in a shallow draft outboard cruiser capable of independent operation from the larger sounding launch. A photograph of such a cruiser is shown in figure 5. This cruiser is also equipped with hull-mounted sounding transducers to permit sonic sounding of shallow areas and a sampling boom to permit sampling, velocity, or nuclear density probe measurements.

The shore station can be transported by vehicle; however, most of the shoreline along the Missouri River reservoirs is rough and irregular, with numerous bays that in many cases make it practically impossible to reach the range line. Transportation in a vehicle eliminates the versatility of being able to select the most desirable side of the reservoir for the shore station. On the other hand, an advantage of vehicle transportation is elimination of the error due to the master station coming close

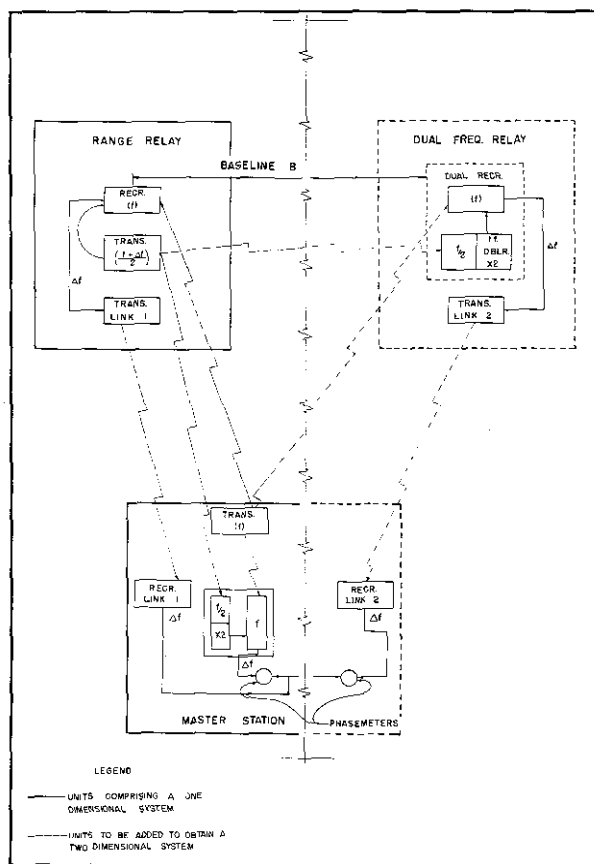


FIGURE 4.—Two dimensional diagram of the type ER Raydist system.

to the shore station (described under test procedure), because the shore station can be set up farther from the shoreline.

Mobile CW Transmitter

For the sounding of long deep ranges, and this pertains to approximately 95 percent of our



FIGURE 5.—Shallow draft cruiser used for independent sounding operations and transportation of the Raydist shore stations.

sounding work, this transmitter is installed with the master station on the sounding launch. The CW transmitter cabinet is fastened in a high frame built for this purpose on the port stern corner. The transmitter's antenna is mounted on the side of the cabinet. Power in this case is obtained from the same 115-volt a.c. generator that supplies power to the master station.

For the sounding of long ranges that have areas that are too shallow for the sounding launch, this transmitter is installed in the out-board cruiser or smaller sounding skiff as necessary. Both boats are equipped with inboard tanks to accommodate the transducers of either an Edo or Bludworth sonic sounder. A 750-watt motor generator supplies power to the Raydist unit while a 12-volt storage battery supplies power to the sounder.

The Master Station

Since the master station serves as the focal point of operations for our Raydist surveys, it and the accessory components are installed on the sounding launch as previously described. The equipment layout varies for different launches, but in general the antenna schematic remains fairly constant as shown in figure 6.

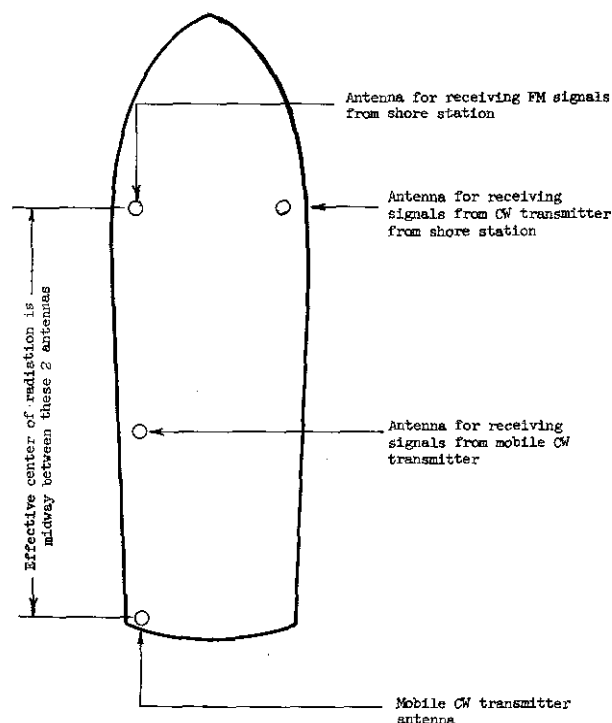


FIGURE 6.—General location for antennas for the master station on sounding launches.

Due to the use of the launch as a base of operations for all deep water observations, some of the components needed for Raydist operations serve dual functions. Perhaps the best illustration of this point is shown on the launch "Ellis" (fig. 7), which was designed specifically for reservoir sedimentation survey work.



FIGURE 7. — Launch *Ellis* on Garrison Reservoir. Only bow booms were installed for this operation.

The "Ellis" is a 38-foot, steel-hulled work launch, propelled by twin diesel engines and containing a small galley, toilet, bunks for three personnel and a fairly large workroom. Other installed equipment includes dual controls, a two-frequency two-way FM radio, AM two-way radio, dial-type depth sounder, and a 3-kw. 115-volt a.c. diesel generator. Both 115-volt a.c. and 12-volt d.c. lights and outlets are installed throughout the boat. A heavy duty convertor is installed on each side to supplement and charge the batteries for each engine from the 3-kw. diesel generator. A flush-mounted power windlass is installed on the bow deck. For convenience in anchoring, 300 feet of $\frac{1}{2}$ -inch nylon line is carried on a shop-made reel. Two booms, equipped with B-50 stream gaging reels, are installed on the bow deck. These booms are used for sampling, for velocity measurements, and for bar checking the depth sounder. The upright members of the booms are used to mount two of the Raydist antennae. Two high booms, one 13 feet and the other 11 feet, are installed on each side near the stern of the boat. (See fig. 8 for typical installation on the launch "Dakota" used at Fort Randall Reservoir.) These booms, equipped with stream gaging reels, are used for taking density samples and for suspending the nuclear sediment density probe. All the booms can be rotated and have a locking device on the base to hold them in several positions.

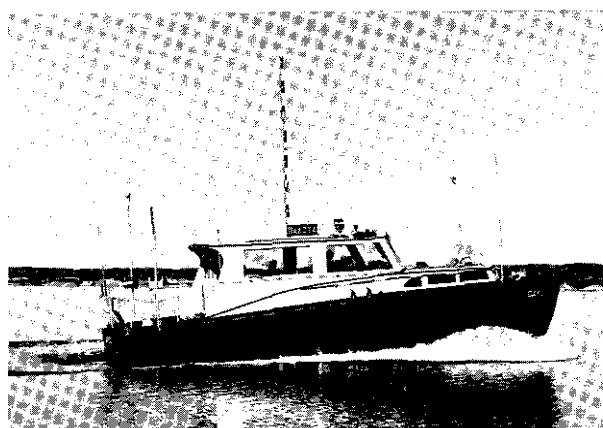


FIGURE 8. — Launch *Dakota* on Fort Randall Reservoir. Note stern sampling booms and antenna arrangement.

A 15-foot triangular stadia board is mounted on top of the cabin. The Raydist master station is installed in a combination rack and table in the front port corner of the main cabin. The Raydist test set and depth sounder are mounted on top or on the table portion of this rack. The remote phase indicator is mounted in a revolving frame on the dash so that it can be turned or oriented for better operational viewing, particularly for the launch operator (fig 9).

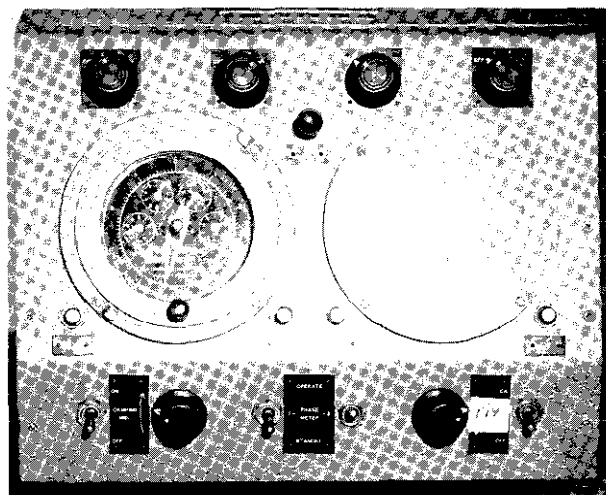


FIGURE 9. — Raydist remote phase indicator. For two-dimensional operation another dial is mounted in place of right cover plate.

Tests to Determine Raydist Performance

Hastings-Raydist, Inc. tested and operated the Raydist system on Chesapeake Bay near its factory in the presence of an electronic technician from the Garrison District. During this

testing, a distance of approximately 20 miles was measured, its repeatability was demonstrated, and distances to several known objects shown on maps of the Chesapeake Bay area were checked.

Prior to receiving the Raydist equipment, three sediment ranges near Garrison Dam and the town of Riverdale, N. Dak., were selected for checking and testing this equipment and lengths were accurately determined by triangulation. For convenience in setting up and checking the Raydist equipment, points were established near the water's edge at both ends of these ranges. During the initial use of the Raydist system, it was discovered that the measured distances could not be checked by the distances obtained with the electronic equipment. Therefore, tests were performed with Raydist to determine the following: (1) The lane length; (2) the effect on the lane length when the master station is close to the shore station; (3) the effect on measurements when the master station is close to bluffs; and (4) the error due to phasemeter lag.

The lane length was checked and determined by running along lines established parallel to the axis of Garrison Dam. These lines were well targeted and flagged at the ends for the boat operator to keep the boat on line. Three transits were set on 90° offsets from the crest movements on the axis of the dam. Whenever the boat passed the line of a transit, flag and radio signals were given for the Raydist operator to read and record the phasemeter reading. Several runs were made along each of the three lines parallel to the axis of the dam. The average of these runs revealed an average lane length of 102.2, which was quite different from the theoretical or computed value of 102.5 feet. The theoretical lane length can be computed by

the equation $\lambda = \frac{c}{f}$; where λ is the wave length,

c —the velocity of propagation of radio waves (299,794 km./sec.),⁵ and f —the radio frequency (4,796.4 kc.) of the basic transmitter. If this equation is used, the computed lane length,

$$\lambda/2 = \frac{(c) (3.2808)}{2f} = 102.53 \text{ feet. This measured or actual lane length was later confirmed by numerous surveys of ranges. It is recognized that corrections for the various factors affecting the propagation of radio waves are necessary to obtain true distances; however, by direct calibration as described above the necessity for determining the various corrections was eliminated.}$$

During our initial surveys with Raydist,

⁵ ASLAKSON, C. I. SOME ASPECTS OF ELECTRONIC SURVEYING. Amer. Soc. Civil Engin. Trans., 117 (Paper No. 2476): 1. 1952.

there was an indication that the lane lengths were shorter when the master station came close to the shore station. To obtain information on this phenomena, stadia and Raydist readings were taken simultaneously, each 50 or 100 feet for 1,000 feet from the shore station. In general, it was discovered that it was necessary to separate the stations 600 feet before this error was eliminated and that most of the error disappeared beyond 400 feet. Closer than 200 feet resulted in substantial errors. Figure 10 shows this error plotted against distance

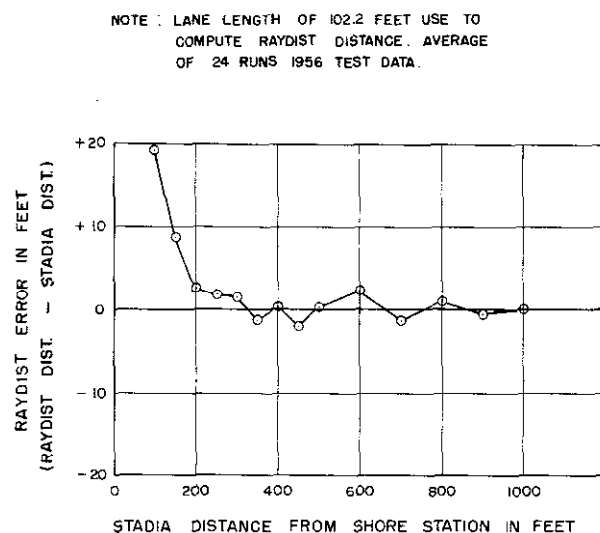


FIGURE 10. — Raydist error when master station is operating near shore station.

from the shore station. To overcome this, it has for the first 1,000 feet of soundings near the been necessary, in most instances, to use stadia shore station. Raydist readings are also obtained at each of these stadia fixes.

Also during some of the early surveys with Raydist, discrepancies were noted in the lane lengths obtained when the master station approached steep bluffs on that side of the reservoir opposite from the shore station. This error was investigated by simultaneously taking stadia and Raydist readings similar to that described above. Investigation was also made by towing the outboard cruiser with the shore station installed and operating. The master station and mobile transmitter were installed aboard the launch doing the towing. The cruiser motors were left down to cause some additional drag and thus increase the tension on the 350 feet of manila towline. The shore station was towed in the open reservoir until the phase meter revealed that the towline was adjusted, i.e., no change in the phase meter reading in-

licated that the length of the towline was not changing. The shore station was then towed past bluffs at varying angles and distances from the shore line. Any deviation of the phase meter now indicated reflection of the radio waves from these obstructions. It was necessary to stay about 300 feet from the bluffs to eliminate reflection. The maximum reflection observed when passing close to steep bluffs was about 0.20 lane. Some reflection was even observed when passing close to low banks approximately 10 feet high. This error is also eliminated by using both stadia and Raydist fixes on the first 1,000 feet of sounding near the opposite shore.

Slight errors due to phasemeter lag were observed during the initial use of Raydist. This error was investigated by taking Raydist readings while making backward and forward runs past the same locations. This error was found to be 0.03 to 0.05 lane. To eliminate this error, lane length ties for both ends of a range must be made with the master station launch pointed and moving in the same direction.

Preparatory Work for Aggradation Resurveys

Preparatory work applies to the survey work that is accomplished by a two- or three-man party prior to the actual sounding of the ranges and includes:

- (1) Installation of the missing markers at the range monuments. The markers consist of 6- by 30-inch white boards fastened to steel fence posts.

- (2) Establishment of control points near the water's edge for both elevation and stationing. If a large rise in the reservoir is forecast, additional control points are established at intervals of approximately 10 to 15 feet in elevation between the water surface and a point higher than that forecast. Flags are set at these control points.

- (3) Profile the end of the range from the water's edge to a point beyond where no change has taken place.

- (4) Determination of the water surface elevation. This elevation is checked against that obtained on the opposite shore and against that recorded by the lake gage. This work saves considerable time for the sounding party since it eliminates doing a substantial amount of survey work and searching for the ranges in cases where the markers have been damaged or destroyed.

Description of Sounding Procedure

Personnel Required

Four to eight personnel can be used to make up the hydrographic party depending upon the

amount of preparatory survey work performed, the amount of sampling, etc., that is required in conjunction with sounding, and other workload items that affect the availability of personnel. Generally, six personnel are used—two at the shore station and four with the master station. The master station party includes a party chief, boat operator, Raydist operator, and sounder operator.

Setting Up the Shore Station

The most desirable end of the range is selected by the shore station party. A location near the range where the cruiser can be beached is preferred. The main duties of the shore station party are: (1) to set up and operate the shore station; (2) to profile the portion of the range between the water's edge and the nearest control point; (3) to run levels to determine the water surface elevation; and (4) to give alinement instructions to the operator of the sounding launch. After going ashore, the party performs the necessary survey work to establish the location for setting up the Raydist units. The units are set normal and about 17 feet on each side of the range line. The offset distance is usually determined by the lengths of the power cord from each cabinet which are plugged into the heavy duty power cord at the range line. The transit is usually set up at the point between the Raydist units for reading stadia and giving alinement instructions to the sounding launch. A portable "handi-talkie" radio is used to give alinement instructions to the boat operator. The shore station AC power supply, instead of the standard battery supply, is usually used for the radio. Figure 11 is a photograph of a typical setup of the shore station units.



FIGURE 11.—Typical arrangement of Raydist shore station units at Garrison Reservoir.

In instances where it is impossible or difficult to reach the shore because of shallow water, bluffs, or vertical banks, the Raydist units are installed on the cruiser. One unit is mounted on the bow while the other is mounted at the stern and the boat is secured across the range line so that the units are equidistant and normal to the line.

Positioning Raydist

It is necessary to tie-in the Raydist phase-meter readings to the range stationing at both ends of the range. When beginning or ending soundings on the *shore station side*, positioning of the Raydist is accomplished by —

- (1) Repeated stadia position checks at 600 or 700 feet from the shore station.
- (2) Chaining to the nearest control point when the Raydist shore station is set over 600 feet from the water's edge, except when shallow water prevents the sounding launch from coming close to shore, in which case it is necessary to use stadia.

When beginning or ending soundings on the *opposite shore side*, the positioning of Raydist is accomplished by —

- (1) Chaining when there is deep water and no bluffs.
- (2) A 600- or 700-foot stadia tie when there is shallow water or bluffs along the shore.

Stadia ties are made with the boat moving very slow and are repeated until two phase-meter readings agree with 0.02 or 0.03 lane length. More than likely, a check reading is also obtained during the sounding near shore when both stadia and Raydist are used.

Sounding of Long Deep Ranges

The shore station party travels to the most adaptable side of the reservoir for setting up the station. Meanwhile, the sounding launch performs a bar check at a deep water location near the range line. As soon as the shore station party has completed setting up its Raydist units, the units are switched from "Off" to "Standby" and the party proceeds with other duties. Personnel on the sounding launch turn on their Raydist units during the bar check so that there is a sufficient (at least 5 minutes) warmup period prior to using the equipment. After completion of the bar check, the shore party personnel switch their Raydist units from "Standby" to "On" and tune the two transmitters. At the same time, personnel on the sounding launch switch the mobile transmitter from "Standby" to "On," tune the transmitter, and check the operation of the entire Raydist system by observing the phase meter and the test set.

When the sounding operation is started from the opposite shore, two personnel and necessary equipment are taken to shore by either the sounding launch or in the case of shallow water by the small boat. These personnel then perform the necessary shore work, position the sounding launch, and if required give stadia fixes to the launch for the 1,000 feet of sounding near shore. One man is left on shore to give alinement to the sounding launch during the main sounding run.

When the sounding is started from the shore station side, the 1,000-foot portion of sounding is accomplished while traveling toward shore; after which, the positioning is accomplished while leaving the shore. At this time, it is necessary for one man from either the sounding launch or the shore station to go to the opposite shore in the small boat in order that he can be setup before the sounding launch arrives. One man must remain on the shore to give alinement to the sounding launch.

The main sounding run (from either shore) is started within the 1,000-foot portion by turning on the depth recorder and the "automatic fix." Position lines are then automatically recorded on the sounding chart for each even lane. The sounding, positioning, etc., at the other end of the range is accomplished in a similar manner to that previously described. Alinement instructions are given the launch operator by radio from either shore. Generally, and sometimes because of radio transmission difficulties, alinement is given for about one-half the range by each transit operator. Also, if some shoreline work remains on either side, the transit operator on the other shore takes care of the alinement duties.

Sounding of Long Ranges with Shallow Spots

Usually on every survey a few ranges are encountered that have areas shallower than the draft of the sounding launch. On Garrison Reservoir these situations are eliminated or at least reduced by surveying as many as possible of the ranges in the backwater reach during low reservoir stages prior to the June rise. If there have been changes (usually deposition) in the overbank, this part of the range profile is later obtained by conventional survey methods. When the launch cannot be used, the mobile transmitter is mounted in one of the smaller boats to obtain the shallow water profile of these ranges. The shore station is set up at one end of the range and the launch containing the master station is anchored at the opposite end of that part of the range to be sounded. Positioning is accomplished in the same manner as previously described. Fixes for the sounding chart are transmitted by radio

from the master station to the small sounding boat.

Use of Calculator

A Monro-Matic calculator is carried on the boat for determining lane lengths and range stationing. The range stationing is placed in the lower dial, the phase meter reading in the upper dial and the lane length in the keyboard. Range stationing for a particular phase meter reading can be obtained by operating the + and — keys so that this meter reading appears in the upper dial. A phase meter reading for a particular stationing can likewise be obtained. The range stations for the sounding “fixes” are determined in this manner and placed on the sounding charts during slack work periods; such as, when traveling between ranges and during periods when one of the master station crew is not busy at other duties. Computations of locations for sampling etc. are also determined with the calculator.

Determination of Lane Length

Lane lengths are determined by dividing the total known distance covered for any given run by the number of Raydist lanes measured for that known distance. Predetermined range stationing is necessary on each shore to provide an overall known distance. In order to eliminate accumulated error from using a predetermined lane length, a lane length is computed and used for each survey of a range. This eliminates prorating any error for the entire survey of a range.

Sampling Procedures

After sounding a range, locations are selected from the sounding chart for taking samples and using the sediment density probe. One observation in the channel and one or two on the overbank are usually adequate. Anchoring of the launch close to the point selected is accomplished by observing the phase meter. After anchoring, samples of the bottom material and density probe readings are obtained. The Foerst sampler is used when the bottom material is “soupy” and not too dense to penetrate. The denser material is sampled by use of a tube-type instrument, sometimes called a “reservoir” or “density” sampler. Density-probe readings are taken at vertical intervals to the penetration limit of the probe. Quite often the crew determines the weight and volume of the samples and saves a small portion in moisture-proof containers for laboratory-size analysis and moisture content determination. On the launch the tube sampler is operated from the high starboard boom, the density probe from the high port boom, the Foerst sampler from a bow boom, and if other samplers, such as a bed

sampler are required, from the other bow boom. At the deepest vertical, temperatures and Foerst samples are quite often taken at intervals of 5 or 10 feet. The temperatures are taken with a resistance-type thermometer to obtain information on the thermal stratification of the reservoir. In general, however, stratification data are obtained during a continuous run up or down the reservoir, so that the information is collected during a shorter time period.

Disadvantages and Limitations of Raydist

The limitations of this equipment must be acknowledged, so that procedures can be devised and prepared to overcome these deficiencies during work in the field. It is recognized that better accuracy would be obtained if we could eliminate the stadia ties when the master station is close to the shore station or bluffs. This could be accomplished by (1) spending additional time to carry the shore station units, motor generator, etc., a sufficient distance inland; and (2) by investigating to discover more about this error and how to adjust for it. Nevertheless, we feel that the accuracy obtained with the use of good stadia ties is satisfactory.

Very few failures of the equipment have occurred during a sounding run; more frequently, trouble is discovered prior to starting the run. Failures during field operations have been minimized by (1) thoroughly checking, several weeks prior to use, the equipment in the shop by a radio technician; i.e., all the tubes are checked and as many as possible of the individual components are checked; (2) having the entire system operated, tested, and adjusted by the radio technician; (3) keeping the motor generator units in good operating condition; and (4) carrying a tube checker, VTM, and a supply of tubes and other spare parts on the sounding launch. Personnel in the master station party are usually experienced in using the testing instruments; therefore, breakdowns can often be repaired in the field without calling out a radio technician.

Cost of Raydist Equipment

At the time of purchase in 1956, a complete type ER Raydist system cost \$42,000. The cost of our single-dimension portion of a type ER Raydist system was about \$27,000.

Since purchase of the Raydist system, Hastings-Raydist, Inc. has developed the type DM Raydist system, which is transistorized to a large extent; therefore, the units are smaller and lighter and require less power. As a result, the shore station can be operated entirely from batteries. This new system measures distance

directly in both dimensions; i.e., to plot position it is only necessary to swing circular arcs using the shore station points as centers.

Conclusions

Various methods and means can be utilized in maintaining horizontal control for the sedimentation survey of reservoirs and each procedure has its own particular advantages or disadvantages. Use of the Raydist-type system of control, however, offers several unique and distinct benefits, which can be advantageously used on almost any sizable reservoir; but, in particular, apply to those larger reservoirs of exceptional width. These advantages include (1) a continuous measurement of varying distances by an accurate, rapid, and convenient method; (2) ability to reposition quickly at

any desired location for additional observations; (3) mobility and versatility of uses; and (4) elimination of a costly triangulation network assuming control by intersection. Disadvantages include high initial cost, some limitations when used near steep bluffs or for the measurement of small distances, and, to a minor degree, the need for the occasional assistance of a trained electronics technician.

The single dimension-type arrangement for Raydist operation is practical, and its characteristics and limitations have been adequately defined through a variety of specific field tests. We were naturally disappointed in discovering certain limitations but are now confident that through our established operational procedures, better than third-order accuracy can be maintained. The practical justification for obtaining this equipment has been well satisfied.

FORECASTING SEDIMENT DISTRIBUTION IN A RESERVOIR BY ELECTRONIC COMPUTER

[Paper No. 87]

By FORREST B. MORROW, *assistant chief, Sedimentation Unit, Hydrology Section, U. S. Army Engineer District, Fort Worth*

Synopsis

Considerable advancement has been made in the past two decades in the field of sedimentation. Basic sediment data have been accumulated by Federal and State agencies in the collection of samples, resurvey of reservoirs, and experimentation in laboratories. Theoretical analyses have been expanded, and the methods of collection, analysis, and interpretation of field data have been improved. With the advent of electronic data processing equipment it became apparent that the long, tedious computations that absorbed many man-hours of work could be programmed for computer analysis and could be accomplished in a fraction of the time previously required. A method for distributing sediment in reservoirs has been developed and programmed for the Model G-15D Bendix general purpose digital computer.

Introduction

It was realized many years ago that sediment would be deposited in a reservoir upstream from any structure that would impede the flow of the sediment-bearing stream upon which it was built. As the streamflow enters the reservoir, the velocity decreases due to the larger cross-sectional area and the larger sizes of sediment begin to settle out. Originally it was assumed that most of the sediment carried into a reservoir would eventually be deposited only in the headwaters area and at the bottom in the main or deepest part of the reservoir. Resurveys of reservoirs have proved this assumption

erroneous and that sediment deposition occurs in various degrees throughout all reaches of a reservoir. The problem then became one of anticipating the sediment distribution at the various levels in a reservoir. Numerous approaches have been made to establish this distribution. As more information became available, the complexity of the influences affecting sediment deposition in a reservoir became more pronounced. The shape of the reservoir, the sediment characteristics, the regimen of inflow, the anticipated reservoir regulation, and the sediment-reservoir volume ratio must be considered each in its proper perspective in the selection of a procedure for predicting sediment distribution in a reservoir.

On studies in connection with investigations for space for sediment deposition in reservoirs within the Fort Worth District it is desirable to know the general trend of the ultimate distribution patterns of this sediment. Various approaches and procedures have been taken in developing methods of predicting the sediment deposition patterns. The method most recently adopted in the Fort Worth District is the empirical area reduction method as presented by Whitney M. Borland and Carl R. Miller.¹ This method was programmed for use on the Bendix G-15D digital computer.

Procedure

In recent years sediment distribution patterns were required in the Fort Worth District for a large number of reservoirs varying in size

and shapes for the various river basins. These patterns were studied in connection with investigations for the U.S. Study Commission—Texas, and for survey reports and detailed plans for construction of numerous Corps of Engineers reservoirs in the district. These studies were time consuming because they also involved investigations of numerous combinations or systems of the already large number of reservoirs. In order to permit meeting schedules, various procedures for estimating distribution of the sediment in reservoirs were investigated. The method described by Borland and Miller¹ was found to be readily adaptable to the various reservoir types in the river basins under investigation.

This procedure requires first that the total amount of sediment anticipated to be deposited in each reservoir be determined and that the reservoir be classified in accordance with the types established. In accordance with the distribution pattern for the type of reservoir under consideration, the elevation of the new or ultimate "zero storage" capacity after the deposition of the estimated volume of sediment is then determined by the mathematical process described in the referenced paper.¹

The amount or quantity of anticipated sediment to be deposited in a reservoir is currently estimated by utilizing procedures recommended by the Texas Water Commission (formerly Texas Board of Water Engineers) and the Soil Conservation Service.² The average annual sediment production rate for the watershed above a given reservoir is determined from the various land resource areas and their appropriate unit rates of sediment yield as established in the publication. The weighted average annual rate of sediment production is then applied to the number of years for which sediment deposition storage space is provided in the project. The elevation of the new zero storage point is then approximated by successive steps through trial and error. This latter procedure is time consuming.

The type of reservoir or curve used to distribute this sediment is established by utilizing the standard type curves presented by Borland and Miller³ as a basis. These curves represent the percentage of sediment deposition versus the relative reservoir depth between the streambed at the damsite and elevation of the reservoir at the maximum controlled storage. The type

of curve that fits the reservoir involved is obtained by relating depth versus initial reservoir capacity with the standard type curves given in the paper. The curves may then be adjusted for anticipated reservoir operation, type of sediment inflow, or capacity/inflow ratio. A specific curve of percentage of reservoir depth versus percentage of sediment deposited (sediment distribution curve) is finally adopted to satisfy the classification of the project under consideration.

Since the last step in this method, that of distributing the sediment, follows mathematical procedures, it is especially adaptable to determination by electronic computer. As presently programmed, the Bendix G-15D computer can determine the elevation of the new zero storage point, distribute the predetermined amount of sediment, and print out the original and revised area and capacity tabulations in a period varying from 20 to 30 minutes, depending upon the number of points in the original area and capacity tabulation and the precision required. The same computations if done manually would generally require from 4 to 8 hours.

The basic data required for the determination of the distribution of sediment in a reservoir by the adopted procedures consist of the original area and capacity tabulation, the amount of sediment to be distributed, the maximum reservoir level below which the sediment is to be distributed (usually top of controlled storage), the desired precision (usually 1 percent of the total amount of sediment to be distributed), and the reservoir type as classified above. The equation for standard type of distribution curves is part of the basic computer program. With these data the computer will determine by trial and error the following elements of the computations within the predetermined degree of accuracy desired:

- (1) The revised area tabulation at successive increments of elevation.
- (2) The revised capacity tabulation corresponding to the area data above.

At first an approximation of the elevation of the new zero storage point is determined by the computer using a trial and error procedure. With this approximate new zero elevation, the computer determines a revised area-capacity table and compares it with the amount of sediment to be distributed for the desired accuracy, repeating the computation until an elevation for the new zero storage point is obtained which is within the limits of desired precision. Once the final elevation of the new zero storage point has been determined, the computer prints out the following elements:

- (1) Elevation;
- (2) Original area;

¹ BORLAND, W. M., and MILLER, C. R. DISTRIBUTION OF SEDIMENT IN LARGE RESERVOIRS. Amer. Soc. Civil Engin. Proc. 84, HY2 (Tech. Paper 1587). 1958.

² SOIL CONSERVATION SERVICE. INVENTORY AND USE OF SEDIMENTATION DATA IN TEXAS. Tex. Bd. Water Engin. Bul. 5912. 1959.

³ See footnote 1.

- (3) Original capacity;
- (4) Revised area;
- (5) Revised capacity.

In a basinwide investigation where numerous combinations and sizes of reservoir projects are studied and more than one distribution of sediment is made for a proposed reservoir, it is desirable to have the original and the revised area and capacity typed out on the same form for ease in identification. The program, as written for the Bendix G-15D computer, is limited to 50 elevation intervals. The elevation intervals may be set in either uniform or varying increments as desired, so long as the total number of intervals does not exceed 50.

The practice in the Fort Worth District has been to start at the topmost elevation of controlled storage and vary the intervals so that in the vicinity of the estimated elevation of the new zero storage point, the elevation interval is one foot. Once the new zero storage elevation has been determined and the computations completed for the initial 50 elevation intervals, the basic computation may be subdivided in order to obtain more detailed information at smaller intervals. During the early stages of its devel-

opment, the Fort Worth Office of the Soil Conservation Service of the U.S. Department of Agriculture also utilized this program and facilities of the Corps of Engineers for estimating the distribution of sediment in reservoirs they were studying.

It is estimated that the current cost of operation of the Bendix G-15D computer based on approximately 100 percent utilization will not exceed \$25 per hour.

Conclusion

The use of the Bendix G-15D computer for sediment distribution calculations has proved satisfactory in the Fort Worth District Office on the basis of man-hours saved in making the tedious computations and on the basis of making qualified engineers available for other studies or investigations that are not readily adaptable to computer programming. A review of an entire river basin can be made by the use of the computer in a fraction of time previously required for the manual distribution of sediment in the reservoirs. As the requirements arise, it is anticipated that many more sediment computations can and will be adapted to the computer programming.

DEVELOPING SEDIMENT STORAGE REQUIREMENTS FOR UPSTREAM RETARDING RESERVOIRS

[Paper No. 88]

By A. F. GEIGER, *geologist, Engineering and Watershed Planning Unit, Soil Conservation Service, U.S. Department of Agriculture, Milwaukee*

Abstract

Problems that are peculiar to the development of sediment storage requirements for upstream retarding reservoirs are discussed.

The wide variation in soil, topographic, and cover conditions that are possible between small watersheds, even within a physiographic region or major land resource area, leads to extreme variations in sediment yields. Measured sediment yield data for a specific watershed are usually lacking, and the time involved between planning and construction is too short to allow for the collection of meaningful data. Where sediment yield data are available, it must be modified to reflect the influence of the application of soil conservation measures.

To meet the specific policy requirements of the Soil Conservation Service pertaining to sediment capacity in retarding reservoirs, two methods are in general use. One involves the transposition of measured data from comparative watersheds. The second involves the estimation of watershed erosion and sediment delivery ratios, used when transposable data are unavailable.

Allocation of storage capacity for sediment accumulation within the reservoir basin requires consideration of the trap-efficiency, specific weight of sediment, and distribution of sediment within the basin. These considerations are discussed briefly.

General

Floodwater retarding reservoirs in small watersheds are designed to provide full effectiveness for a minimum period of 50 years. To assure the full effectiveness for the designed economic life, additional capacity must be provided in the reservoir to offset the depletion of capacity due to the accumulation of sediment within the reservoir.

The volume of storage that must be allocated to sediment is dependent upon the sediment yield from the watershed, the trap efficiency of the reservoir, the volume-weight of the deposited sediment, and, to a certain extent, the distribution of sediment within the basin. The most important of these is sediment yield.

Sediment yield can be defined as the total

amount of sediment that is delivered to or transported past a given point in a watershed. It is made up of that part of the products of erosion within the watershed which escape redeposition upstream from the point of measurement. In other words, sediment yield is dependent upon the total amount of erosion within the watershed (gross erosion) and the proportion of the products of erosion that is delivered to the point of measurement (delivery ratio). Delivery ratios are usually described as percentage of gross erosion. Since sediment is the product of erosion, factors that influence erosion must necessarily influence sediment yields. There is no direct relationship, however. One watershed may have a high rate of erosion but a low delivery rate, whereas another, with a relatively low ratio of erosion, may have a high delivery ratio.

The trap efficiency of a given reservoir is the ability of that reservoir to retain sediment which is delivered to it. Trap efficiency is the percentage relationship between sediment yield to a reservoir and sediment retained by the reservoir. It is dependent upon numerous factors, such as detention storage time, sediment particle characteristics, type of spillways and inlets, reservoir shape, and quality of water.

The volume-weight (dry weight) of sediment deposited within a reservoir may vary from less than 30 pounds per cubic foot to over 100 pounds per cubic foot. An overall average for measured volume-weights in existing small reservoirs would be in the neighborhood of 60 to 65 pounds per cubic foot. The ultimate volume that a given weight of sediment will occupy within a reservoir will depend upon its texture and mineral composition and whether it is permanently submerged or is alternately submerged and aerated.

The distribution of sediment in a reservoir is dependent upon the physical characteristics of the sediment, the shape and underwater topography of the reservoir, the nature of the channel above the reservoir, and the time required to empty the retarding pool. Capacity must be provided for all sediment that is deposited below the elevation of the emergency spillway.

Sediment Yields

At present the best known method of determining sediment yield is the long-term sampling of suspended sediment and bedload and recording of rainfall and runoff data on the watershed. Land-use data taken during the period of record may indicate the effects of land-use changes on sediment yield. These data combined with expected future land-use changes,

can then be used to make a reasonable prediction of future sediment yields. Unfortunately, such data are practically never available for small watersheds where structures are being planned. On occasion, an adjacent watershed having similar physical characteristics may have sediment yield data available. These data can often be transposed to the watershed in question, making adjustments for differences in drainage area and land use.

The most substantial source of measured sediment yield data comes from sedimentation surveys of existing small reservoirs. This provides a composite record of sediment yield from the drainage area above the reservoir for the period since the reservoir was built. However, it is usually impossible to isolate the effects of individual runoff events or to determine the effects of land-use changes. Periodic resurveys can sometimes overcome this difficulty.

The third alternative, when neither sediment load data nor reservoir survey data are available, is to determine rates of gross erosion and sediment delivery ratios. This involves the identification of the sources of sediment, computation of the volume of sediment available from each source, and an analysis of the factors that influence delivery rates.

Sediment Sources

A sediment source study is made to locate the origin of the sediment, to determine what part of the sediment is derived from each source, and to determine the effects of land treatment upon the production of sediment. The primary sources of sediment are sheet erosion, which is the removal of soil by the combined effects of raindrop impact and shallow overland flow, and channel erosion, which is the removal of soil by concentrated flow. Channel erosion includes gullying, streambed and bank erosion, and flood plain scour. There may be other sources of sediment within a local watershed, such as gravel washing plants or quarries.

Sheet erosion

Sheet erosion implies the removal of soil material from the land surface by overland flow. It is initiated by the impact energy of raindrops. Qualitatively, the primary factors influencing the rate of sheet erosion are, as summarized by Musgrave:¹

- (1) Rainfall, characterized particularly by intensities and amounts in the determination of the energy of impact;
- (2) Flow characteristics of surface runoff, particularly as affected by slope degree and slope length;
- (3) Soil characteristics, particularly those physical properties that affect erodibility; and

¹ MUSGRAVE, G. W. THE QUANTITATIVE EVALUATION OF FACTORS IN WATER EROSION—A FIRST APPROXIMATION. Soil and Water Conserv. Jour. 2 (3): 133-138. 1947.

- (4) Vegetal cover, characterized by comparative densities and protective effects.

Data that bear upon the relation of these major factors to soil erosion were assembled from approximately 40,000 storms on fractional acre plots at 20 erosion experiment stations. A group of U.S. Soil Conservation Service research specialists was brought together to analyze these data. The results of the analysis were reported by Musgrave.¹ From the analysis the following equation was developed, which can be used for estimating the rate of sheet erosion under variable conditions:

$$E = IRS^{1.35} L^{0.35} P^{1.75}$$

where

E —soil loss, in inches;

I —inherent erodibility of the soil, in inches;

R —a factor describing cover conditions;

S —degree of slope, in percent;

L —length of slope, in feet;

P —maximum 30-minute intensity rainfall of a 2-year frequency, in inches.

The general procedure of the use of this empirical equation has been outlined by Gottschalk.²

More recently, the data used by Musgrave and additional data that have become available since the original analysis were subjected to another analysis. The result was the development of an improved soil loss equation. The general procedures for the use of this equation, which relate the same major factors to rates of soil erosion, have been outlined by Wischmeier and Smith.³

In both equations the vegetal cover and the use of conservation practices are shown to have a most profound effect upon land erosion. The rate of sheet erosion for clean-tilled crop or fallow land will be upwards of one hundred times the rate of erosion for established grass meadow, all other factors being equal.

Public Law 566 requires that not less than 50 percent of the lands situated in the drainage area above each retention reservoir must be under agreement to carry out recommended soil conservation measures and proper farm plans. In addition, not less than 75 percent of the effective land treatment must be installed, or their installation provided for, on those sediment source areas which, if uncontrolled, would require a material increase in the cost of construction, operation, or maintenance of the

structural measure. These additional requirements must be scheduled for installation either before or concurrently with the installation of the structural measures.

This, then, necessitates the computation of gross erosion rates under "present" conditions, while soil conservation measures are being applied or are becoming effective, and under "future" conditions, when the soil conservation measures have become fully effective.

The average annual rate of sheet erosion under present conditions is computed from existing soil-survey and land-use data, supplemented by field reconnaissance where necessary. The total rate of sheet erosion on a watershed is determined by the summation of the sheet erosion on individual, more or less homogeneous increments. This is the rate that is used until the end of the project period, at which time all required land-treatment measures will have been applied and become effective.

The average annual rate of sheet erosion which will prevail after the installation of required land-treatment measures is also computed. This computation takes into account the effectiveness of better crop rotations, strip-cropping, stubble mulching, contouring, terracing, land conversion, and controlled grazing in the control of erosion. This is the rate that is used for the remainder of the project period. The Department of Agriculture, in cooperation with the local sponsoring organization, has the responsibility of determining what these required land-treatment measures are, and the local sponsoring organization has the responsibility of applying the measures to the land.

Channel erosion

Channel erosion implies erosion by concentrated flow and is influenced mainly by such factors as channel gradient, channel soil characteristics, hydraulic parameters, and sediment supply and characteristics. In channel stability studies, these factors must all be considered. However, existing criteria are inadequate for estimating the expected volume of channel erosion under the highly variable conditions which occur within a watershed. Consequently more generalized methods have been developed by the Soil Conservation Service. This involves the field measurement of the net enlargement of channels over a known period of time, the time factor being determined through the use of aerial photographs of different ages and by interview of the local people. The average annual rate of channel erosion is then applied to the project period.

Other sediment sources

Other sources of sediment that occur within a watershed must be studied individually and control methods evaluated. The effects of urban-

² GOTTSCHALK, L. C. PREDICTING EROSION AND SEDIMENT YIELDS. Internatl. Union Geodesy and Geophys., Gen. Assembly 11, 1 (tome 1): 269-276. 1958.

³ WISCHMEIER, W. H., and SMITH, D.D. SOIL-LOSS ESTIMATION AS A TOOL IN SOIL AND WATER MANAGEMENT PLANNING. Internatl. Assoc. Sci. Hydrology, Comm. on Land Erosion Pub. 59, pp. 148-159. 1962.

ization on sedimentation rates are of special interest. Only limited studies have been conducted to date, but these studies indicate the seriousness of the problem brought on by the rapid expansion of urban construction.

Sediment Delivery Ratios

Ideally, the delivery ratios of sediment from each of the various sources of erosion should be determined and applied to compute sediment yields. However, determining the source of sediment after it reaches the point of measurement (sediment samples of reservoir surveys) is usually impossible, and delivery ratios must therefore be determined on an overall basis.

The determination of sediment delivery ratios when both sediment yield and gross erosion are known is a simple matter of arithmetic. When sediment yields are not known and only gross erosion can be computed, it is necessary to estimate delivery ratios so that sediment storage requirements can be computed.

The relationship of sediment delivery ratios to measurable influencing factors, such as the

magnitude of sediment sources, climate, soils characteristics, topography and other watershed characteristics, and depositional environments, has been analyzed by several workers in the field of sedimentation. The results of several of these analyses have been reported by Roehl.⁴ Significant relations appear to exist between delivery ratios and area of drainage, average total stream length, and relief-length ratio. Local and regional analyses have been made, and curves have been developed in most areas of the country which relate delivery ratios to these physical factors. These curves are used locally in the estimation of expected future sediment yields from small watersheds.

Trap Efficiency of Reservoirs

The trap efficiency of a reservoir is the percentage relation between the amount of sediment that is delivered to a reservoir and the amount that remains within the reservoir. It is a function of detention storage time, character of sediment, type and location of spillways, reservoir shape, quality of water, and probably several other factors. An analysis conducted by Brune⁵ indicates a close relation between trap efficiency and the ratio of the capacity of a reservoir to the average annual inflow. This relation, as used by the Soil Conservation Serv-

⁵ BRUNE, G. M. TRAP EFFICIENCY OF RESERVOIRS. Amer. Geophys. Union Trans. 34 (3): 407-318. 1953.

⁴ ROEHL, J. W. SEDIMENT SOURCE AREAS, DELIVERY RATIOS AND INFLUENCING MORPHOLOGICAL FACTORS. Internatl. Assoc. Sci. Hydrology. Comn. Land Erosion Pub. 59, pp. 202-213. 1962.

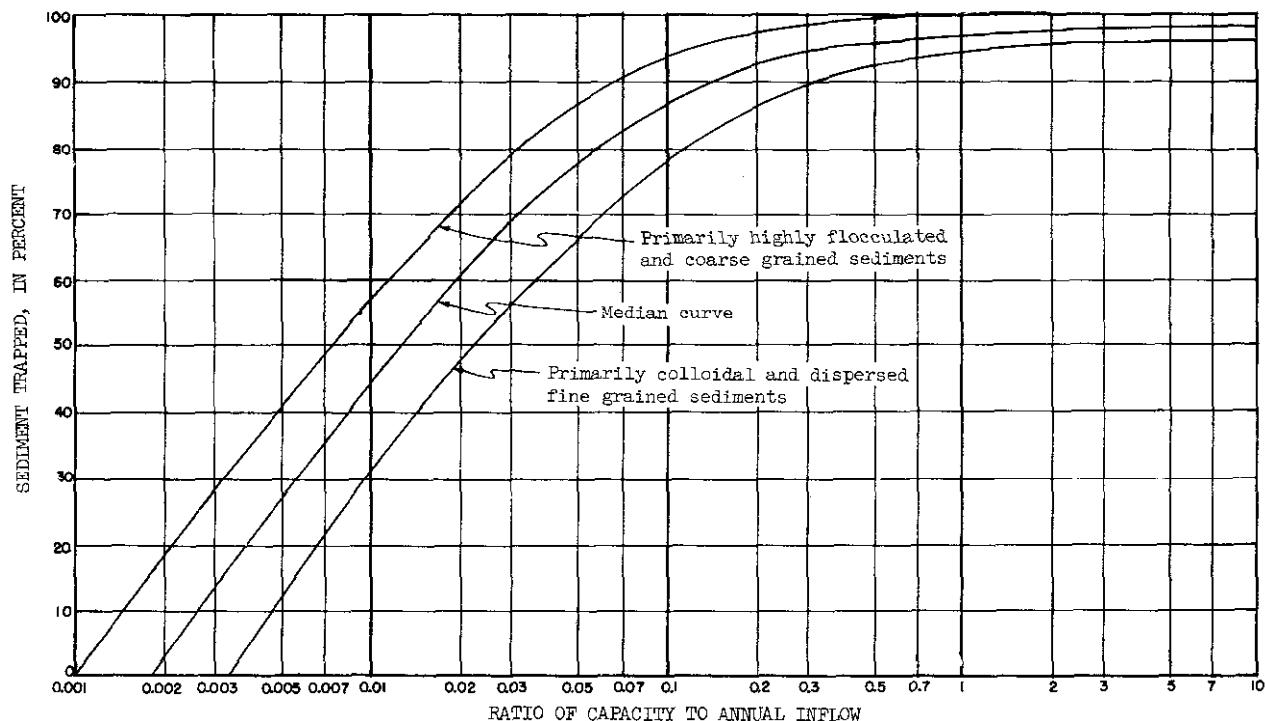


FIGURE 1. — Trap efficiency of reservoirs. (Adapted from Brune.)

ice, is illustrated by figure 1. These curves allow for the modification of trap efficiency estimates based on the expected character of sediment. Capacity inflow ratios in small detention reservoirs are seldom less than 0.1 and trap efficiencies are seldom less than 90 percent.

Volume Weight of Sediment

The ultimate volume occupied by deposited sediment will depend essentially upon its texture and whether it is permanently submerged or is subject to alternate submergence and aeration. Where possible, volume-weight analyses are made of sediments in existing reservoirs to aid in the prediction of volume weights in proposed reservoirs. Because it is not always possible to make field measurements, table 1 has

TABLE 1.—*Indicated ranges in volume-weight of reservoir sediment*

Dominant grain size	Permanently submerged	Aerated
	Lb./cu. ft.	Lb./cu. ft.
Clay.....	40 to 60...	60 to 80.
Silt.....	55 to 75...	75 to 85.
Clay-silt mixture.....	40 to 65...	65 to 85.
Sand-silt mixture.....	75 to 95...	95 to 110.
Clay-silt-sand mixture.....	50 to 80...	80 to 100.
Sand.....	85 to 100...	85 to 100.
Gravel.....	85 to 125...	85 to 125.
Poorly sorted sand and gravel...	95 to 130...	95 to 130.

been devised, based on the analysis of available data on mechanical analyses and corresponding volume weights of sediments.

With a knowledge of the character of the inflowing sediment and whether it will be submerged or aerated, an acceptable estimate of the volume weight of sediment in a proposed reservoir can be made with the aid of the table. Since soil in place will have a different volume than the same weight of soil deposited as sediment, it is necessary to make all estimates of sediment yield as weights instead of volumes.

Sediment Distribution

The portion of the incoming sediment that will deposit above the elevation of the principal spillway will vary with the amount and character of the sediment and the stage of the reservoir. In general, a larger part of the coarse-grained sediment will be deposited above this elevation. The general distribution of sediment below the principal spillway, after its elevation has been set, is of little consequence. However, that portion that is deposited above the principal spillway will have a bearing on the elevation of the spillway. A given volume of storage is allocated to sediment. This volume, if all sediment would occupy the lowest parts of the pool, would normally determine the elevation of the principal spillway with the detention pool above this elevation. When a portion of the sediment

is deposited in the detention pool, the elevation of the principal spillway must be lowered by an amount equal to this volume of storage.

Little information is available on sediment distribution in retarding reservoirs, but from an analysis of available data the following guide lines have been developed to aid in estimating deposition above the principal spillway.

(1) In watersheds of low to moderate relief where the predominant sources of sediment are silty and clayey soils, where sheet flow is the principal eroding agent, and the sediment is transported primarily in suspension, approximately 10 percent will be deposited above the principal spillway.

(2) In watersheds of low to moderate relief, with combination sheet and channel erosion, where products of erosion are essentially equal amounts of medium to fine sands, silts and clays, transport of coarser materials is along the bed and finer materials are in suspension, approximately 20 percent will be deposited above the principal spillway.

(3) In watersheds of moderately high relief where the primary source of sediment is from channel erosion producing substantial amounts of coarse sand and gravel that are transported as bedload, with smaller proportion of fine-grained sediment transported in suspension, approximately 30 percent.

(4) In watersheds of high relief where the primary sediment load consists of boulders, cobbles, and sand, above 30 percent.

These percentages may be adjusted upward or downward, using judgment based on local watershed conditions.

These estimates are also used to help in determining what portion of the deposited sediment will be aerated and what portion will be permanently submerged. Other factors will also enter into this determination. For instance, drawdown outlets may be provided so that the sediment pool will be "dry." Or seepage and/or evaporation losses may exceed inflow for long periods of time.

Sediment Storage Requirements

The foregoing procedures and computations are used by the Soil Conservation Service in the determination of sediment storage requirements and the allocation of this volume to various portions of the reservoir. The Soil Conservation Service is continually striving to improve its estimates and to develop new tools and procedures. The collection and analysis of data that is becoming available as the small watershed program advances will aid in the refinement of estimates and improvement of procedures. Continuing research in the field of sedimentation is doing much to this end.

A NEW METHOD OF ESTIMATING DEBRIS-STORAGE REQUIREMENTS FOR DEBRIS BASINS

[Paper No. 89]

By FRED E. TATUM, *chief, Hydrology and Reservoir Regulations Section, U.S. Army Engineer District, Los Angeles*

Synopsis

A method was needed for computing the debris production for all areas for which debris basins are to be built. It is recognized that there are many factors affecting debris production. The method presented in this paper was based on observed data, most of which resulted from floods that occurred when the ground was conditioned for runoff by prior rain and from areas that had been partially or completely burned 1 to more than 10 years prior to the flood. The observed debris amounts were adjusted to a common base and curves were developed by trial and error to represent separate adjustments for the major factors affecting debris production. This procedure is then adapted to estimating debris production for areas where debris basins are to be built.

Introduction

The design debris capacity for debris basins in the Los Angeles area of the Los Angeles District originally was based on a flat rate of 50,000 to 100,000 cubic yards per square mile of drainage area, depending upon the results of an inspection of the area. Later, when more information had been compiled, the debris basin capacity was based on an enveloping curve of debris inflows plus 100 percent (fig. 1) and a field inspection of the drainage area. It was realized that a more detailed method of determining the debris capacity for use in the design of debris basins was required to afford equal protection to areas below debris basins. To meet this requirement a new method of computing debris production from a drainage area was developed. The method is based on evaluating the main factors that affect debris production. The development and use of the method are presented in this paper.

Available information on debris deposition at alluvial cones and on debris inflow at reservoirs and existing debris basins in the Los Angeles area, as well as in generally comparable areas in the Southwest, was evaluated. It was recognized that in the production of debris, several factors are involved, including the size and shape of the drainage area; the steepness of canyons and side slopes; geological characteristics (such as type of rock and soil, and weathering effects); the type and density of plant cover; recency of burns; and the frequency, the duration, and, in particular, the intensity of storms and floods.

Debris-Production Factors

A review was made of all available reports on debris, and after a study of the records of major debris flow in the Los Angeles County area, the factors that were found to have the most effect on debris production are as follows: (1) drainage area, (2) slope, (3) drainage density, (4) hypsometric-analysis index, (5) 3-hour rainfall, and (6) burn effect. Definition of the factors as used in this paper are given in the following paragraphs.

Drainage Area

The drainage area is the area upstream from the considered concentration point or debris-basin site, in square miles.

Slope

The slope is the average slope of the longest watercourse upstream from the considered concentration point or debris-basin site, in feet per mile.

Drainage Density

The drainage density is the total stream length in miles, divided by the drainage area in square miles. The total stream length is the summation of stream lengths of all streams indicated by those streamflow lines upstream from the considered concentration point or debris-basin site that are shown as either intermittent or perennial on the pertinent U.S. Geological Survey map. (1/24,000 scale U.S. Geological Survey maps were used in this study.)

Hypsometric-Analysis Index

The hypsometric-analysis index is based on the hypsometric analysis developed by W. B. Langbein and others and presented in U.S. Geological Survey Water Supply Paper 968-C, dated 1947. The analysis involves the development of a nondimensional elevation-area curve with unity equaling total height and total drainage area above the considered concentration point (fig. 2). Arthur N. Strahler, in an article titled "Quantitative Analysis of Watershed Geomorphology," published in the December 1957 "Transactions, American Geophysical Union," states that the hypsometric curve properties tend to adhere generally to the same curve family for a given geological and climatic combination.

The hypsometric-analysis index, as developed by the Los Angeles Engineer District, is the relative height at which the drainage area is divided into two equal parts (see fig. 2, relative

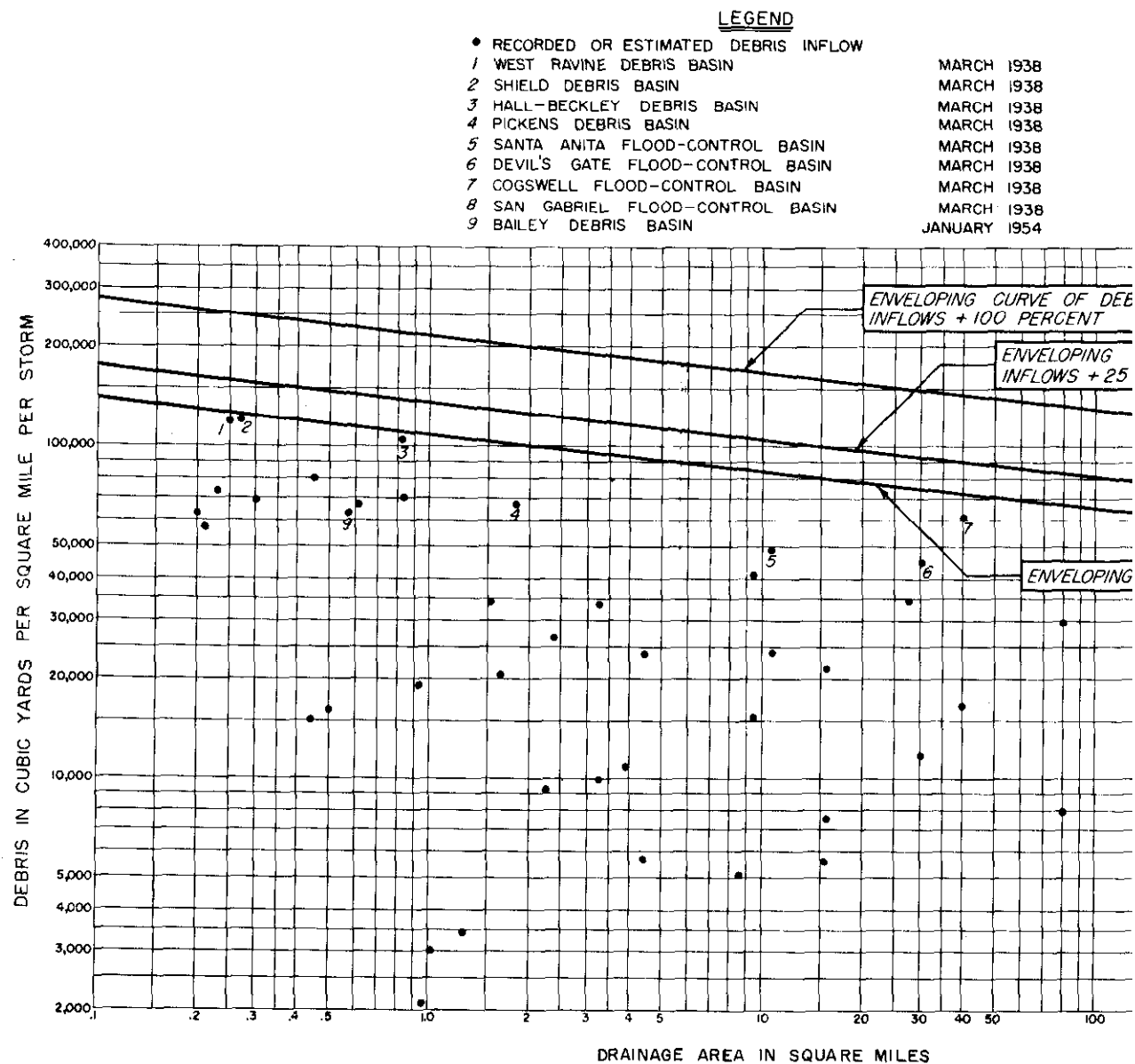


FIGURE 1. — Debris inflows, debris, and flood control basins in Los Angeles County, Ca

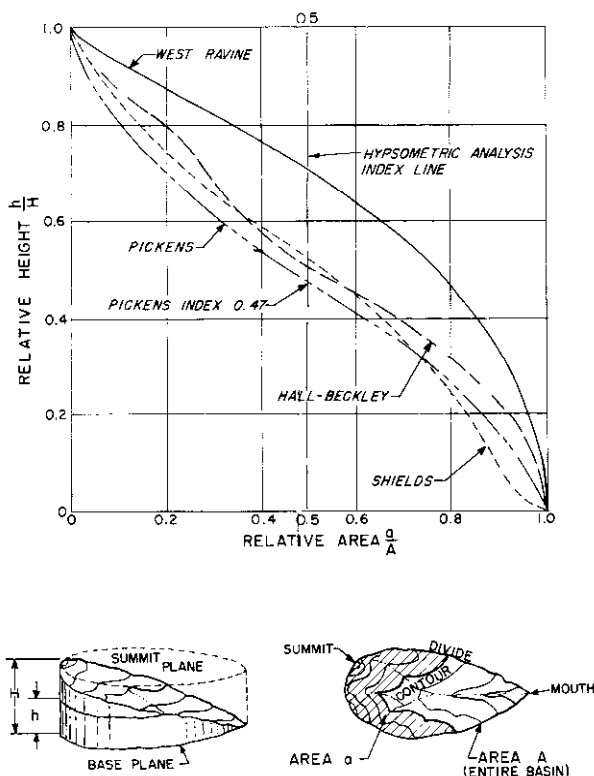


FIGURE 2.—Hypsometric curves for drainage areas above selected debris basins in Los Angeles County, Calif.

area equals 0.5). A comparison of the index numbers with the geology for the drainage areas studied indicated that soil most likely to erode was generally in drainage areas with the index number equal to or less than 0.5. At the same time, index numbers less than 0.5 reflect the lessened debris potential affected by the concentration points being farther removed from the steeper part of the drainage area. Index numbers larger than 0.5 indicate soils generally less likely to erode. The relation of the hypsometric-analysis index to an analysis of the drainage area by a geologist is shown in table 1.

Three-Hour Rainfall

The 3-hour rainfall in inches is the maximum 3-hour rainfall occurring during the storm after prior rain has conditioned the ground for debris movement. This 3-hour period of rain was determined to be the critical period of rainfall for use in determining the effect of rain on debris production. The area that produced most of the debris quantities used in this study during the 1938 flood were conditioned for debris production by rain prior to the maximum 3-hour rainfall.

Burn Effect

The burn effect depends on the percentage of drainage area burned and the recency of the burn.

Observed Debris Production

Data on debris flow have been obtained from (1) a paper by B. H. Dodge titled "Design and Operation of Debris Basin," published in proceedings of the Federal Interagency Sedimentation Conference, Denver, Colo., and dated January 1948; and (2) reports and unpublished data of Los Angeles County Flood Control District. Pertinent data and observed debris production are given in table 2 and the locations of the debris basins are shown on figure 3.

Adjusted Observed Data

To determine the effects of the various factors on production of debris, it was necessary to adjust the observed data to a common base. This was done by adjusting the observed debris production to the amount that would have been produced if the storm had occurred the first year after 100 percent burn in the drainage area. The adjustment was based on data from the report by California Forest and Range Experimental Station, Forest Service, U.S. Department of Agriculture, titled "Hydrologic Analysis Used to Determine Effects of Fire on Peak Discharge and Erosion Rates in Southern California Watersheds," by P. B. Rowe, C. M. Countryman, and H. C. Storey, and dated February 1954. Hereafter in this report, that report will be referred to as the U.S. Forest Service report.

The observed debris production from a drainage area (A) can be represented by the equation:

$$X_o = X_{10} A_n + X_v A_b \quad (1)$$

Where X_o = Observed debris production, in cubic yards;

X_{10} = Rate of debris production 10 or more years after burn, in cubic yards per square mile;

X_v = Rate of debris production for "v" year (1 to 10) after 100 percent burn, in cubic yards per square mile;

A_n = Portion of drainage area not burned, in square miles; and

A_b = Portion of drainage area burned, in square miles.

The relation of debris production rate in percentage of first year production and 100 percent burn to the debris production rate for "v" years (1 to 10) after the burn, based on data from the U.S. Forest Service report, is given on figure 4. The debris production rate for 10 years after a burn, in percentage of first year and 100 percent burn, is 3 percent. It is realized

TABLE 1.—*Relationship of the hypsometric-analysis index to the analysis of the areas by a geologist*

Debris basin or site	Drainage area	Hypsometric-analysis index	Geological analysis	Debris basin or site	Drainage area	Hypsometric-analysis index	Geological analysis
Brand.....	<i>Square miles</i> 1.13	0.50	Brand Canyon drainage basin is composed of closely fractured granitic and metamorphic rocks with thin soil and sparse to moderate growth of chaparral. Steep side slopes composed of such rock form a definite potential for high debris production.	Wilson.....	<i>Square miles</i> 2.54	0.55	Wilson Canyon drainage area is similar to that of other San Gabriel Mountains front canyons for about two-thirds of the drainage area and forms a definitely high potential for debris production. The debris potential of the rest of the area appears to be much less.
Childs.....	.36	.46	Childs Canyon drainage basin is composed of closely fractured granitic and metamorphic rocks with a thin soil and a sparse to moderate growth of chaparral. Steep side slopes composed of such rock form a definite potential for high debris production.	Schoolhouse	.28	.59	Schoolhouse Canyon drainage area is a definitely high potential for debris production from two-thirds of the area where the material is similar to other San Gabriel Mountains front canyons. The debris potential of the remaining one-third of the area overlying sedimentary rocks is only moderate.
Sunset.....	1.11	.46	Sunset Canyon drainage basin is composed of closely fractured granitic rocks with thin soil and sparse to moderate growth of chaparral. Steep side slopes composed of such rock form a definite potential for high debris production.	Live Oak...	.29	.65	Live Oak Canyon drainage area below Live Oak Dam is about 40 percent terrace area of gently rolling hills and about 60 percent more or less steep canyon slopes. The terrace area has a low debris potential, whereas the canyon portion has a moderate to high debris potential.
Elmwood...	.34	.44	Elmwood Canyon drainage basin is composed of closely fractured granitic and metamorphic rocks with thin soil and a sparse to moderate growth of chaparral. Steep side slopes composed of such rock form a definite potential for high debris production.	Emerald....	.19	.65	Emerald Wash drainage area is about 75 percent terrace area with a low debris potential and about 25 percent steep canyon slopes with a moderate to high debris potential.
Blanchard..	.50	.40	Blanchard Canyon drainage basin's geological factors affecting debris potential are comparable to those of nearby Haines Canyon; the debris potential is high.	Emerald (East)	.53	.72	Emerald Wash (East) drainage area is about 70 percent terrace area with a low debris potential and about 30 percent steep canyon slopes with a moderate to high debris potential.
Hillcrest...	.41	.33	Hillcrest Canyon, and its right tributary (Sherer Canyon) in particular, has a drainage basin of closely fractured granitic and metamorphic rocks with thin soil and sparse to moderate growth of chaparral. Steep side slopes composed of such rock form a definite potential for high debris production.	Lopez.....	1.56	.82	Lopez Canyon drainage area has a shallow soil cover and conditions are such that debris contribution should be substantially less than at areas along the mountain front to the east where debris production has been measured.

that the relationship developed in the U.S. Forest Service report was based on annual rainfall and may not be a true relationship for a single storm. However, this is a relative matter, in that all developed curves of separate adjustment factors are interrelated; therefore, a deviation from the relationship presented on figure 4 would tend to be compensated for in the development of the other curves. The relation of

debris produced in a given year after a burn to the debris production percentage rate for that year is equal to the relationship for 10 years after, as in the following equation:

$$\frac{X_v}{C_v} = \frac{X_{10}}{3}$$

or $X_v = \frac{C_v X_{10}}{3}$ (2)

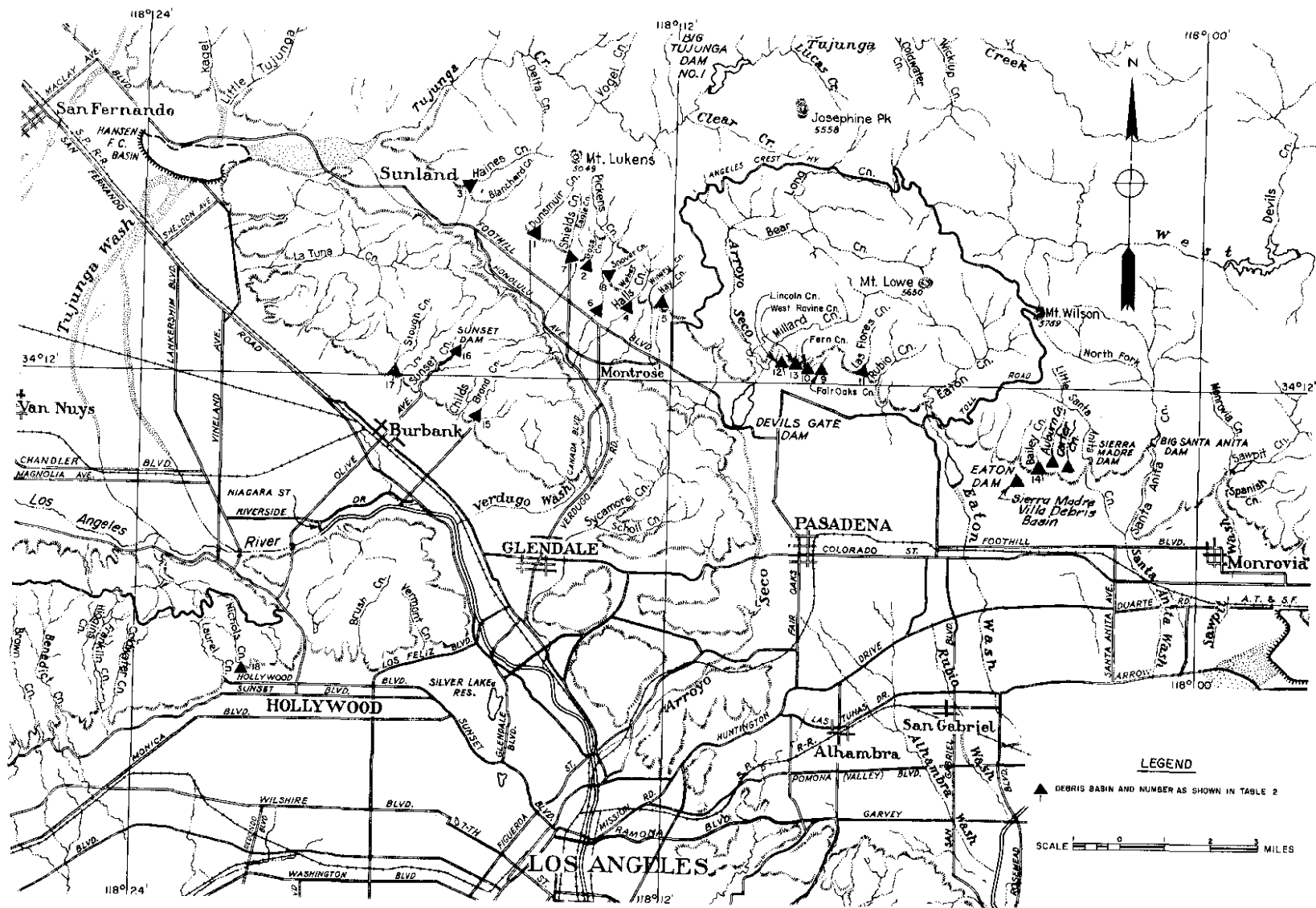
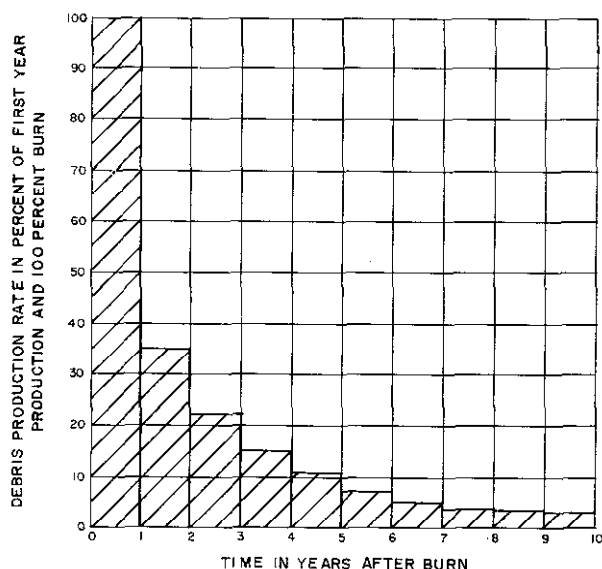


FIGURE 3. — Location of debris basins.

TABLE 2.—Observed data and computed debris production for selected debris basins in the Los Angeles area¹

Debris basin ²			Burn in drainage area		Debris-producing flood		Observed debris production during flood		Observed debris rate adjusted to 100 percent burn 1st year ³	Debris production factors for—				Correction factors					Computed debris production	
No.	Name	Drainage area	Year	Area	Year	Years after burn	Total	Rate		Slope	Drainage density	Hypso-metric index	3-hour rain-fall	Slope	Drainage density	Hypso-metric index	3-hour rain-fall	Total	For maximum 1 square mile with 100 percent burn 1st year ³	For year of observed flood and actual area burned ²
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
	La Cresenta Area:	<i>Square miles</i>		<i>Square miles</i>			<i>Cubic yards</i>	<i>Cubic yards per square mile</i>	<i>Cubic yards per square mile</i>	<i>Feet per mile</i>	<i>Miles per square mile</i>		<i>Inches</i>	<i>Percent</i>	<i>Percent</i>	<i>Percent</i>	<i>Percent</i>	<i>Percent</i>	<i>Cubic yards</i>	<i>Cubic yards</i>
1	Dunsmuir...	0.84	1933	0.78	1938	5	58,800	70,000	695,000	1,390	1.7	0.54	2.94	99	97	98	67	64	1,220,000	106,000
2	Eagle-Goss...	.61	1933	.46	1938	5	40,900	67,050	767,000	1,480	3.3	.25	2.89	100	84	33	64	18	342,000	19,400
3	Haines...	1.53	1933	1.01	1938	5	52,000	33,990	423,000	1,040	2.4	.46	2.85	92	95	98	62	52	1,010,000	117,000
4	Hall-Beckley	.83	1933	.63	1938	5	86,300	103,980	1,185,000	930	2.2	.50	2.82	88	96	100	61	59	990,000	73,000
5	Hay.....	.20	1933	.06	1938	5	12,600	63,000	1,190,000	1,290	.6	.65	2.72	97	99	76	56	40	760,000	9,700
6	Pickens.....	1.84	1933	1.75	1938	5	122,200	66,410	650,000	940	3.4	.47	2.93	88	82	99	67	48	910,000	160,000
7	Shields.....	.27	1933	.24	1938	5	33,500	124,000	1,320,000	1,570	2.5	.51	2.90	100	94	100	65	61	1,160,000	34,400
8	Snover.....	.23	1933	.11	1938	5	16,800	73,040	1,110,000	1,280	3.5	.54	2.82	97	81	98	61	46	874,000	15,500
	Pasadena Area:																			
9	Fair Oaks...	.21	1935	.21	1938	3	12,000	57,140	257,000	1,180	0	.21	2.34	95	100	25	36	9	171,000	9,600
10	Fern.....	.30	1935	.30	1938	3	20,700	69,000	310,000	1,180	4.8	.41	2.42	95	51	92	39	17	323,000	24,600
11	Las Flores...	.45	1935	.31	1938	3	36,000	80,000	495,000	1,610	3.2	.58	2.62	100	85	94	49	39	741,000	60,000
12	Lincoln.....	.50	1935	.50	1938	3	8,000	16,000	72,000	780	4.6	.31	2.39	81	57	60	38	11	209,000	25,600
13	West Ravine	.25	1935	.24	1938	3	29,800	119,200	555,000	1,290	0	.40	2.40	97	100	90	38	33	627,000	39,500
14	Bailey.....	.58	1953	.46	1954	1	65,000	112,070	140,000	1,520	2.6	.50	1.70	100	93	100	12	11	209,000	104,000
	Burbank Area:																			
15	Brand.....	1.03	1927	.77	1943	10	3,100	3,010	99,000	910	3.0	.50	1.70	87	88	100	12	9	171,000	5,300
16	Sunset.....	.44	1927	.42	1938	10	6,600	15,000	495,000	1,610	2.0	.48	1.93	100	97	99	18	17	323,000	4,800
17	Stough.....	1.65	(⁴)	(⁴)	1943	(⁴)	33,500	20,300	670,000	1,020	4.4	.62	2.64	91	63	85	52	25	475,000	20,000
	Beverley Hills Area:																			
18	Nichols.....	.94	(⁴)	(⁴)	1938	(⁴)	17,900	19,040	626,000	480	.9	.56	2.48	57	99	96	42	23	437,000	12,600

¹ See fig. 3 for location.² Computed by use of equation 5 (see text) and adjusted for size of drainage area.³ 1,900,000 times total percent from column 19.⁴ 10 years or more assumed to have no effect on debris production.



NOTE:
CHART BASED ON U.S. FOREST SERVICE REPORT (SEE TEXT).

FIGURE 4. — Debris production rates following burn.

Where C_v = debris production rate for "v" year (1 to 10) after burn, in percentage of rate for first year and 100 percent burn.

Equation 2 is substituted in equation 1:

$$X_o = X_{10} A_n = \frac{C_v X_{10} A_b}{3}$$

$$\text{or } X_{10} = \frac{3X_o}{3A_n + C_v A_b} \quad (3)$$

Also from figure 4 we get the relation:

$$\frac{X_1}{100} = \frac{X_{10}}{3}$$

$$\text{or } X_1 = \frac{100}{3} X_{10} \quad (4)$$

Where X_1 = rate of debris production first year after 100 percent burn, in cubic yards per square mile.

If equation 3 is substituted in equation 4:

$$X_1 = \frac{100X_o}{3A_n + C_v A_b} \quad (5)$$

By means of equation 5 the observed rates of debris production were adjusted to first year after 100 percent burn and are given in table 2, column 10. The adjusted values were then plotted versus drainage area in square miles (fig. 5). A curve was then drawn through the two highest points (parallel to the enveloping curves shown on figure 1, which reflects the effect of the size of drainage area on debris production) to represent the ultimate debris-potential index of production the first year after 100 percent

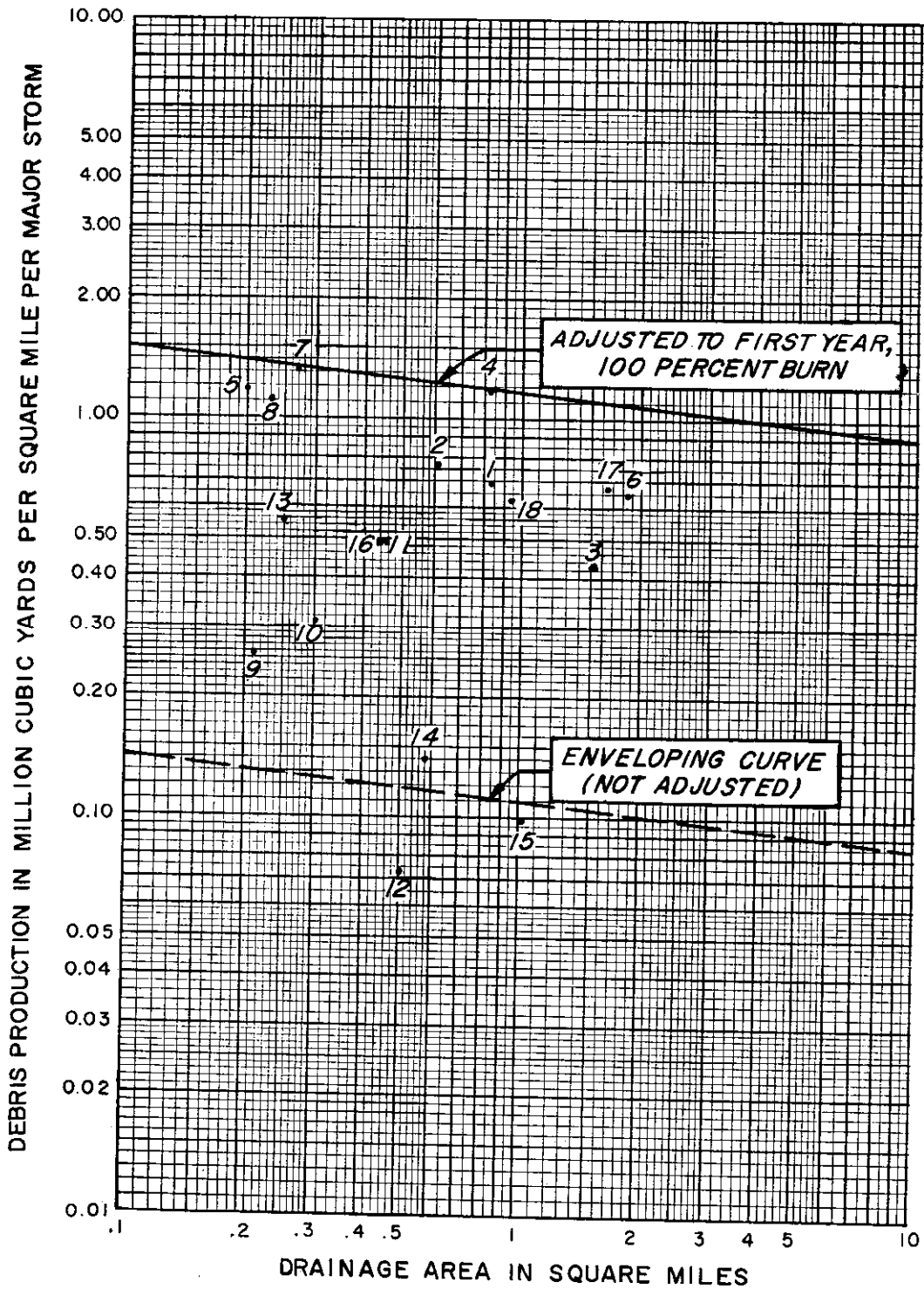
burn for a 3-hour rain of approximately 2.9 inches. This curve represents maximum debris from the areas with the more severe debris producing characteristics.

Development of Adjustment Curves

The development of method was based on the assumption that the debris potential for an area was equal to the product of the ultimate debris-potential index and the percentage values for each of the four factors (slope, drainage density, hypsometric-analysis index, and 3-hour rainfall) representative of the area. It was considered that each factor, if not the ultimate for the area, would reduce the debris potential index by a percentage representing the difference between the ultimate and actual. Curves were drawn to represent the relationship for each of the four factors. These curves were then adjusted by trial and error to give the best results when computed data were compared to observed data. The adjusted curves are shown on figures 6 and 7.

The steps used in computing debris production, for comparison with the observed values, are as follows: (1) the slope, drainage density, hypsometric-analysis index, and 3-hour rainfall for the debris-producing storm are determined for each area; (2) the percentage correction for the above factors are determined from the applicable curves on figures 6 and 7; (3) the percentage corrections are multiplied together to give a total correction; (4) the total correction is multiplied by 1,900,000 cubic yards, which — as shown on figure 8 — represents the ultimate debris-potential index for 1 square mile in maximum-debris-producing areas, to give the maximum debris production in cubic yards from areas of 1 square mile; (5) the value thus computed for maximum debris production in cubic yards for areas of 1 square mile is then adjusted for size of area under consideration by plotting the computed value on figure 5, drawing a curve through the plotted point and parallel to the enveloping curve, and reading the value for the area under consideration from this line; and (6) the adjusted value is then corrected to the actual area burned and number of years after the burn by use of equation 5. The results thus obtained are given in table 2, column 21.

The comparison of the observed and computed debris production is shown on figure 9. The curve shown on figure 9 represents computed values equal to observed values and indicates that the relationship of computed to observed values are reasonable. The scatter of the points is partly due to (1) the difference in ground conditions prior to the flood, (2) the variation in ground cover, (3) the probable



NOTE :

NUMBERS INDICATE DEBRIS BASINS AS LISTED IN TABLE 2
AND DATA WERE TAKEN FROM COLUMNS 3 AND 10.

FIGURE 5.— Enveloping curves of observed debris production adjusted to 100 percent burn first year, for one major storm.

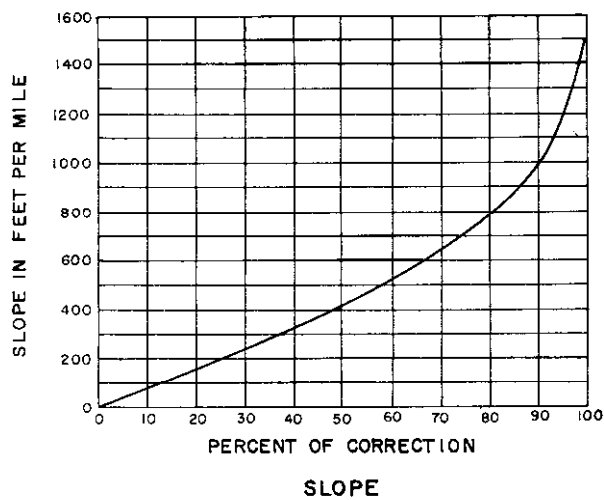
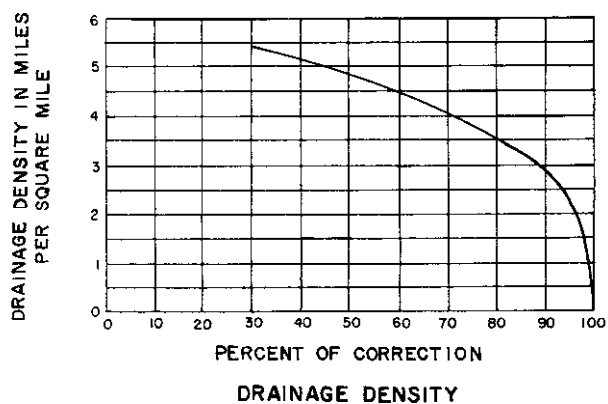


FIGURE 6. — Drainage density and slope curves indicating correction of debris production.

error in determining observed debris quantities and 3-hour rainfall amounts, and (4) the possible inconsistency in using the streamflow lines in determining total stream length to be used in the determination of drainage density.

Verification of Method

Verification of the method is indicated by comparing results obtained by computing the amount of debris produced during the storm of 7 to 12 February 1962 in the Los Angeles area, with amounts actually produced according to the Los Angeles County Flood Control District measurements. A fire occurred in the drainage areas of Sierra Madre Villa, Bailey Canyon, Auburn Canyon, and Carter Canyon debris basins (fig. 3) in October 1961. A storm occurred over these areas on 7 to 12 February

1962, with a total rainfall of about 11 inches. The maximum 3-hour rain occurred in the evening of the 10th. Although prior to this storm the ground conditions were relatively dry, the rain prior to the maximum 3-hour rain was reasonably adequate to condition the ground for runoff. Pertinent information for drainage areas above the debris basins and the computed and measured amounts of debris are given in table 3.

Recommended for Design

The design capacity of a debris basin should represent that amount of debris for one major storm that would be exceeded in magnitude only on rare occasions. The assumption that a major storm would occur, with ground conditions con-

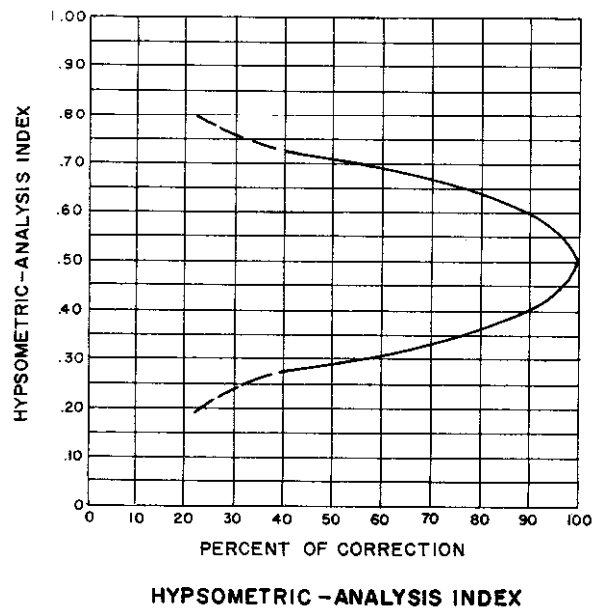
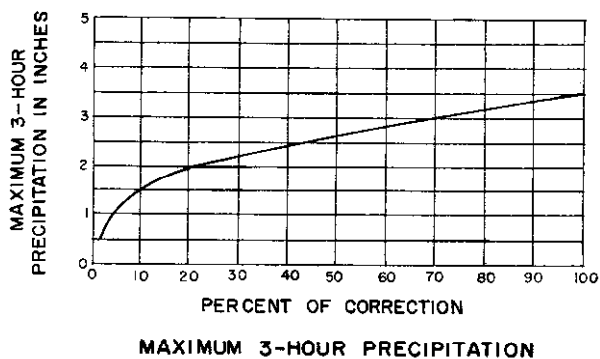
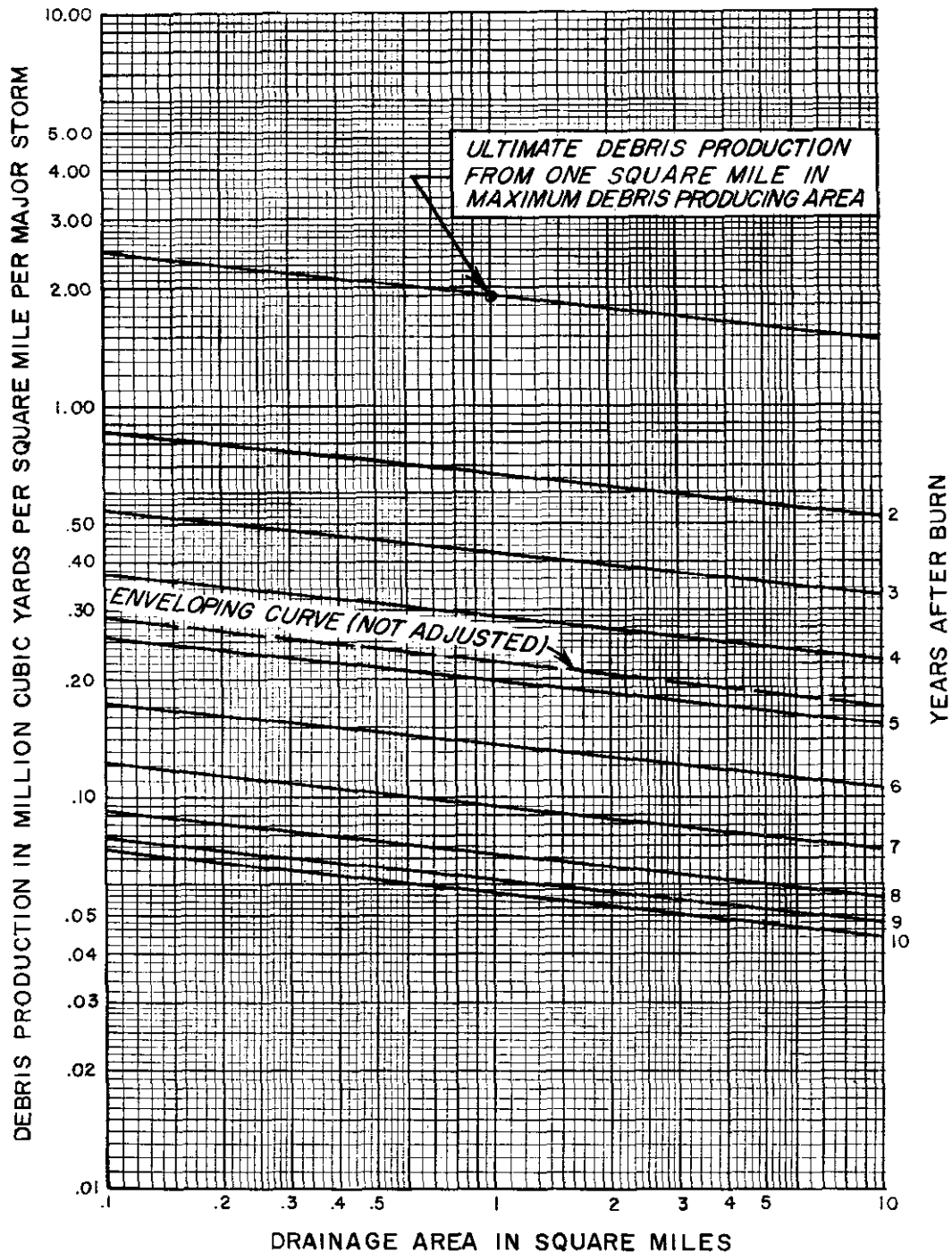


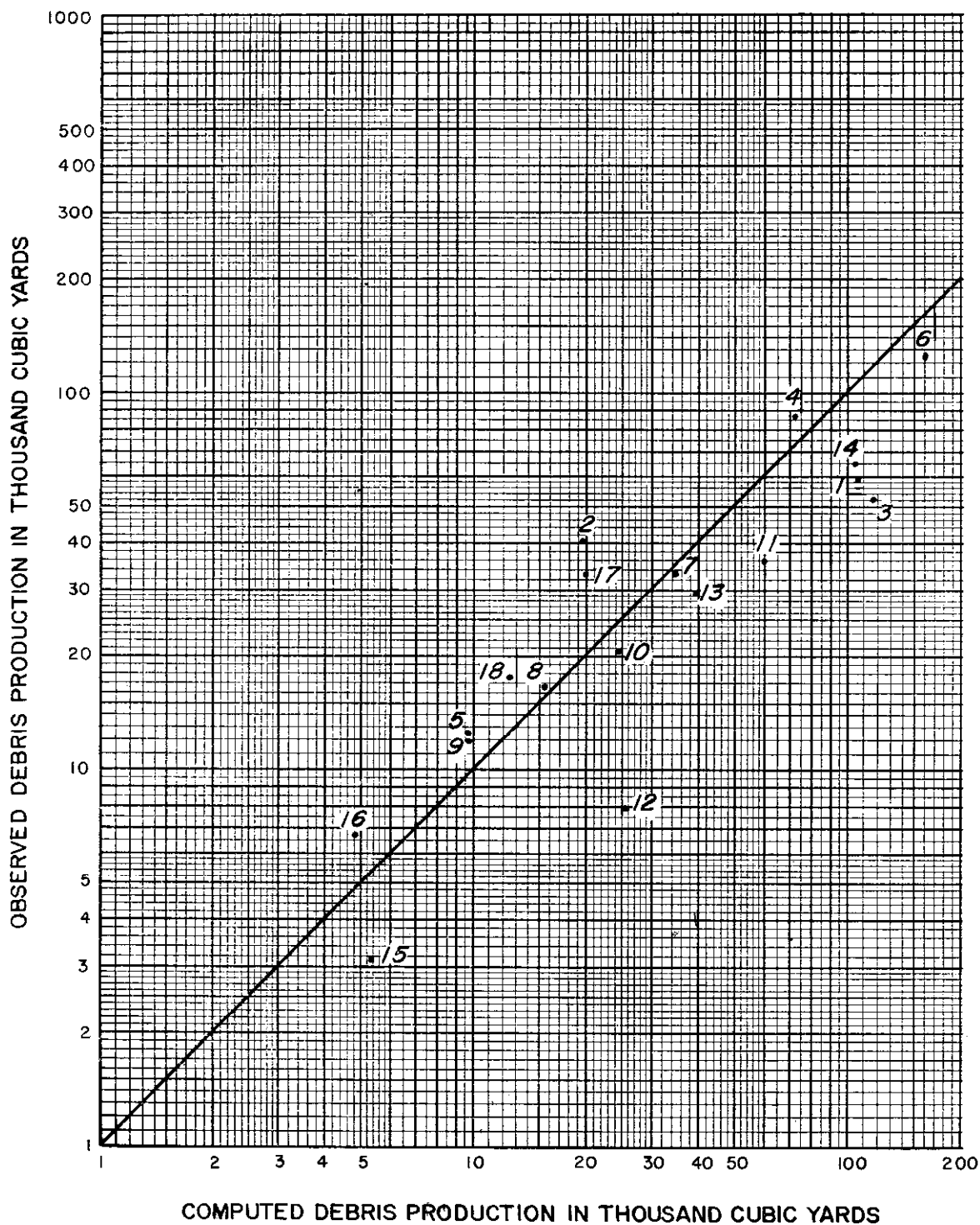
FIGURE 7. — Maximum 3-hour precipitation and hypsometric-analysis index curves indicating correction of debris production.



NOTE:

CURVES (BASED ON DATA FROM FIGURES 4 & 5) REPRESENT ULTIMATE DEBRIS PRODUCTION FOR INDICATED NUMBER OF YEARS AFTER 100 PERCENT BURN WITH ALL CORRECTION FACTORS ASSUMED AS 100 PERCENT.

FIGURE 8.— Ultimate debris production from maximum debris-producing area for one major storm.

**NOTE:**

NUMBERS INDICATE DEBRIS BASINS AS LISTED IN TABLE 2 AND DATA WERE TAKEN FROM COLUMNS 8 AND 21.

FIGURE 9. — Debris production relationship.

TABLE 3.—Observed data and computed debris production from storm of 7 to 12, February 1962, Los Angeles area

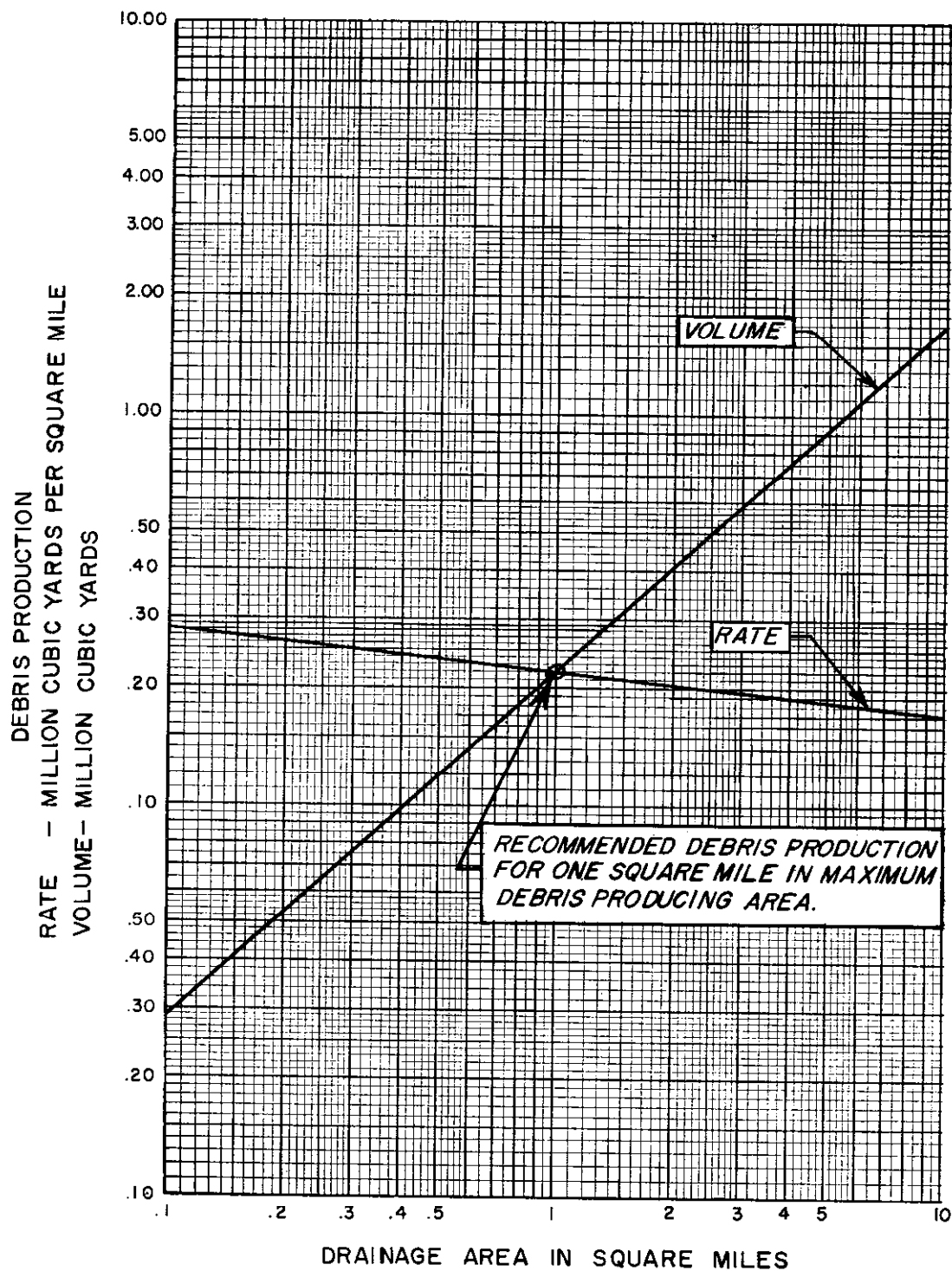
Debris basin ¹	Drainage area	Area burned	Observed debris production	Debris production factors				Correction factors					Computed debris production ²
				Slope	Drainage density	Hypsometric index	3-hour rainfall	Slope	Drainage density	Hypsometric index	3-hour rainfall	Total	
	Square miles	Percent	Cubic yards (3)	Feet per mile	Miles per square mile		Inches	Percent	Percent	Percent	Percent	Percent	Cubic yards
Sierra Madre Villa.....	1.6	36	10,000	1,160	2.2	0.55	1.10	95	96	97	5	4.4	48,000
Bailey Canyon.....	.58	26	10,000	1,525	3.5	.62	1.10	100	80	85	4	2.7	9,300
Auburn Canyon.....	.21	80	20,100	2,240	0	.52	1.20	100	100	99	6	6.0	22,200
Carter Canyon.....	.11	100	11,700	2,540	3.5	.62	1.40	100	80	85	8	5.5	12,400

¹ See fig. 3 for location.² Computed as explained in text under paragraph titled "Development of Adjustment Curves."³ Amount of debris produced not available. Total of 122,450 cubic yards of debris has accumulated since completion of the debris basin in February 1958.

ductive to runoff during the first year after 100 percent of the drainage area was burned, is believed to be too severe for design purposes. As indicated on figure 4, the debris potential of an area decreases rapidly in the first 5 years after a burn. The recommended design debris quantity from a maximum debris producing area of 1 square mile is 220,000 cubic yards, or about 12 percent of the maximum index value, unadjusted. This value is equivalent to the value for 1 square mile area obtained from the enveloping curve of debris inflow (fig. 1) increased by 100 percent. This value also represents the amount of debris that would be produced from a similar area if a major storm occurred, with ground conditions conducive to runoff, 4 to 5 years after 100 percent of the drainage area was burned (figs. 8 and 10). This value is then adjusted by the factors affecting the debris potential for the drainage area being studied and the resulting computed value is the capacity for design of the debris basin in question.

Conclusions

The method presented in this paper is based on using varying degrees of debris potential for the drainage area above each debris basin to determine such debris capacities for use in design of debris basins as will provide approximately equal protection to the different downstream areas. The method is believed to give reasonable values within the limits of presently available data. Obviously, where many interrelated variables are involved, a large number of basic data are required for definitive analysis. Further studies will be made as data become available. The method presented herein is believed applicable for other than the Los Angeles area; however, the correction factors should be verified. One of the basic assumptions in developing the method is that ground conditions, prior to the maximum 3-hour rainfall, are conducive to runoff. In order to use this method of computing debris potential for areas with various prior ground conditions, development of additional correction factors would be necessary.



NOTE:

CURVES SHOWN HEREON ARE BASED ON THE ASSUMPTION THAT THE DEBRIS PRODUCING STORM OCCURS 4 TO 5 YEARS AFTER 100 PERCENT BURN IN THE AREA.

FIGURE 10. — Recommended debris production from maximum debris-producing area for one major storm.

MEASUREMENT OF SEDIMENT DENSITY WITH GAMMA PROBES

[Paper No. 90]

By J. ROGER MCHENRY, soil physicist, and PAUL H. HAWKS, geologist, USDA Sedimentation Laboratory, Agricultural Research Service, Oxford, Miss.¹

Introduction

Sedimentation studies generally require that the density of the sediment deposits be determined in order that estimates of deposition and accumulation can be made. The conversion of a measured volume of accumulated sediment to a weight basis is dependent on this information. Many types of soil and sediment samplers have been developed for taking undisturbed cores for volume-weight measurements. Numerous improvements have been made in the undisturbed core-type samplers, but their operation in underwater sediments has not been completely satisfactory (4).

Nuclear techniques have been employed in recent years for measuring the density and/or thickness of numerous materials (5, 6). The single-probe technique, which utilizes the reflection principle, has been successfully adapted to the measurement of soil density (2, 9), and of sediments in situ (1, 3, 7). This method has one disadvantage, which may, or may not, be serious, depending on the objectives of the particular study. The single-probe system measures the attenuation of the gamma rays, which, after emission from a radioactive source, are reflected or scattered back to the probe by the surrounding medium. The direct transmission of these gamma photons from the source to the detector tube must not occur, and is prevented by placing a lead shield between the source and the detector. In order to shield effectively the detector tube from the radioactive source, a shield of about 6 inches of lead is required. This physical separation of the source and detector in the vertical axis limits the density measurement to an ellipsoidal volume with a vertical axis of 12 to 18 inches. Thus, the determination of densities of narrow bands or near interfaces of sediments is not possible (7). In many cases this is not, however, a serious limitation of the method.

Van Bavel and coworkers (10) utilized the attenuation of gamma rays that passed through a medium between the source and detector to measure soil density. This procedure is known as the transmission method. The transmission method requires two access tubes or probes: one for the radioactive source, one for the de-

tector. With proper instrumentation it is possible to measure the density of thin or narrow bands of material in situ. The transmission method of measuring density and thickness has been used extensively and successfully in industry (5, 6). The possibilities of using such a dual-probe system for the measurement of sediment densities in situ have been investigated and reported by this laboratory (8). This paper reports on the development of the dual-probe system and compares its performance in the laboratory and in the field with that of the single probe.

Methods and Materials

Gamma Probe

The single probe, henceforth called a gamma probe, used in this study was manufactured by Technical Operations, Inc.,² according to specifications furnished by the Beach Erosion Board (1). The original probe has been modified. This modification was largely one of streamlining the exterior container so that insertion into and removal from sediments was expedited. The gamma probe as used is shown in figure 1, together with necessary or desirable accessory equipment.

This particular probe employs a 3-millicurie radium-226 source located near the tip. The source is separated from the three Geiger-Mueller detector tubes by 6 inches of lead. The GM tubes are connected to a transistorized preamplifier, and from this signals are fed through a cable to a portable scaler, Nuclear-Chicago Model 2800.

Dual Probe

The dual-probe system, henceforth designated as the dual probe, consists of a gamma source, a scintillation detector, a fixed dual access-tube assembly for spacing the probe and source, and a suitable power supply and electronic read-out device. The equipment is shown in figure 2. The gamma radiation is produced by a 7-millicurie cesium-137 source placed in the smaller access tube. A NaI (Tl activated) scintillation crystal (1 inch in diameter, 1/2 inch high) is used to detect the gamma emissions. The crystal, photomultiplier tube, and transistorized preamplifier are housed in the probe in the larger access tube (2-inch diameter). The distance between the two access tubes is maintained constant, 12-inch center to center, by the jig arrangement shown in figure 2. The power supply and scaler-ratemeter used in this

¹ Research cooperative with the University of Mississippi and Mississippi State University.

² Reference to commercial equipment or manufacturer is in no way an endorsement by the United States Department of Agriculture.

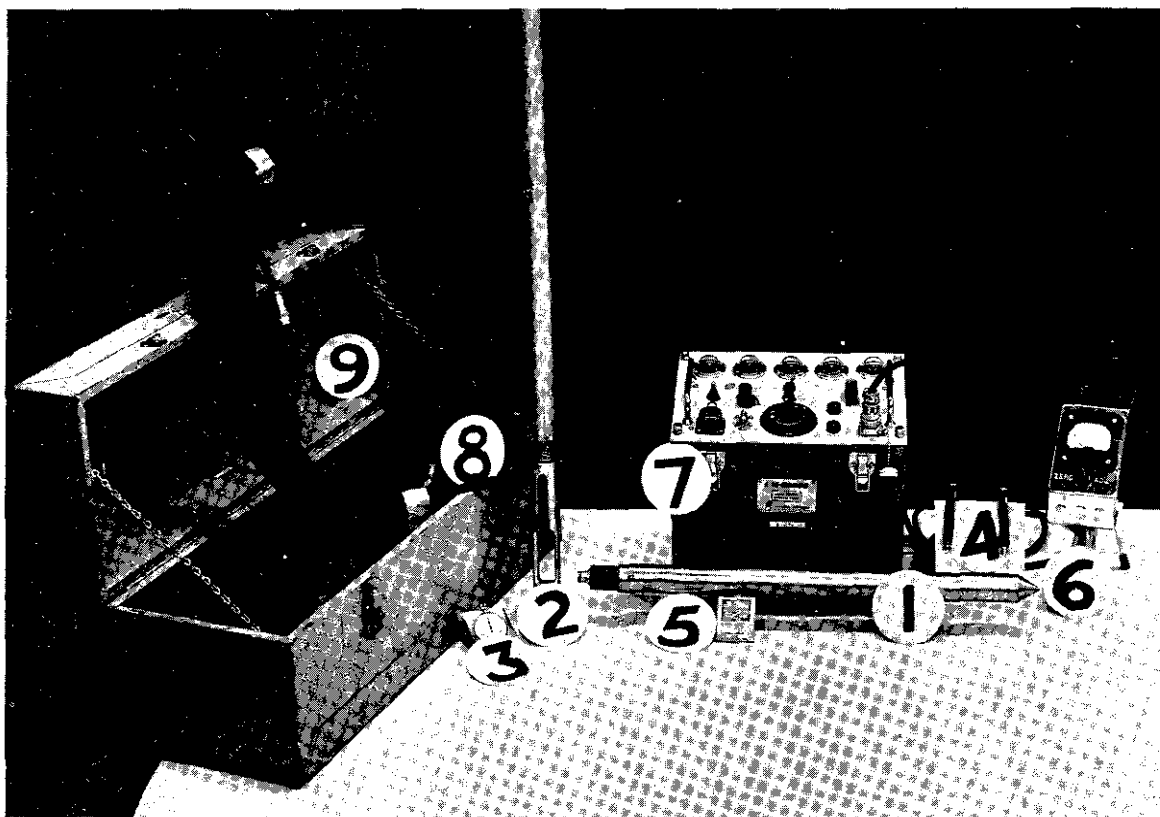


FIGURE 1. — A single gamma probe with operating accessories; (1) gamma probe containing the radioactive source, shielding, Geiger-Mueller detection tubes, and transistorized preamplifier; (2) aluminum connecting sections for inserting probe; (3) stopwatch; (4) personnel exposure pencils; (5) personnel exposure badge; (6) radiation meter; (7) portable scaler; (8) connecting cable; (9) carrying box, with lead shield, for gamma probe.

study was a Troxler Model 200B unit. This instrument is transistorized and when used in the field employs a silver-cadmium dry battery that can be recharged readily. In the laboratory this instrument can be operated from standard alternating current or from the battery. The Nuclear-Chicago scaler used with the gamma probe can also be operated in the laboratory from an alternating current supply.

Theory of Operation

The attenuation of a beam of gamma photons (rays) in matter follows the exponential relationship:

$$I = I_0 e^{-\frac{\mu x}{\rho}} \quad (1)$$

where I is the emergent intensity observed for an incident radiation intensity, I_0 , in a material whose linear absorption coefficient is μ , its thickness x , and of density ρ . The absorption coefficient of a material is the product, $N\sigma$, of the number of atoms, N , per unit volume and of the cross section, σ . The mass absorption coefficient

is $N\sigma/\rho$. The linear absorption coefficient, μ , is equal to $N\sigma$. In this paper μ is used as the total linear absorption coefficient. If μ and x are constant; that is, if the medium is homogeneous and geometry or distance is constant, the ratio I/I_0 is inversely proportional to the density, ρ .

The exponential equation 1 is valid only when the emissions are monoenergetic and when the medium through which the emissions pass is homogeneous; that is, a constant μ . It is theoretically possible to obtain proportional values of $I_1, I_2, I_3, \dots, I_n$ for values of $\mu_1, \mu_2, \mu_3, \dots, \mu_n$ and sum them for total attenuation, but the procedure would be laborious. Practically, the medium to be tested should be homogeneous so that the necessary calibration information can be obtained readily. The requirement for monoenergetic emissions may be handled in either of two ways. Shielding of the source and detector may be employed so a minimum of scattered or reflected radiation reaches the detection system along with the unattenuated primary emission. In order to screen out effectively the undesired radiation, bulky and heavy shielding is re-

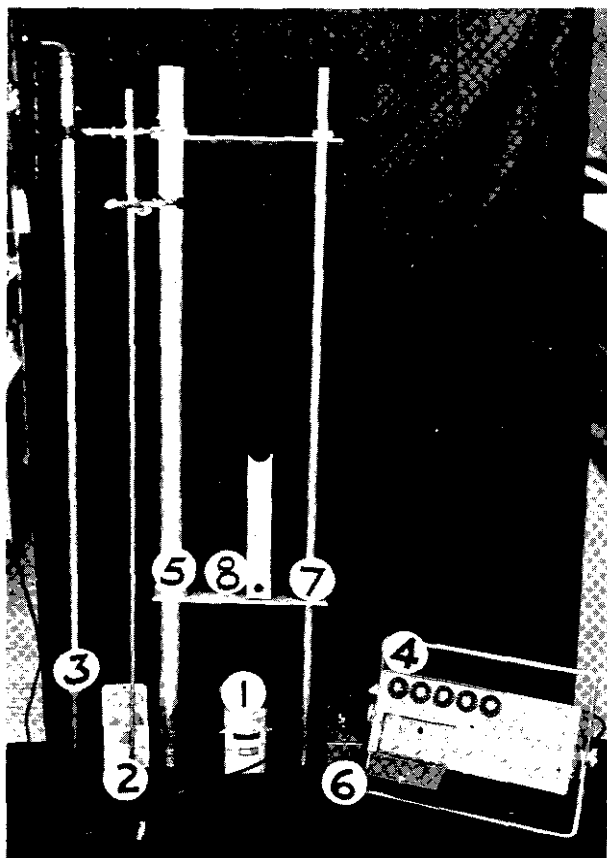


FIGURE 2.—A dual-probe assembly and accessories necessary for operation: (1) lead container for storage of radioactive source when not in use; (2) aluminum rod for inserting source in access tubing; (3) detector probe containing scintillation crystal, photomultiplier tube, and preamplifier and attached to a connector rod for insertion in access tube; (4) portable, transistorized, scaler-ratemeter; (5) aluminum access tube, 2-inch diameter, for detector probe; (6) battery charger; (7) aluminum access tube, $\frac{3}{8}$ -inch diameter for source; (8) standard magnesium bar for calibration, and resting on aluminum spacer for maintaining correct alignment of access tubes. Personnel exposure devices are not shown.

quired. This requirement presents problems in the field use of this method of controlling radiation detection. It is possible also to screen out unwanted radiation by the use of gamma scintillation spectrometry. All emissions below a certain energy level are discriminated against and are unrecorded when the scintillation spectrometer is operated integrally. Effectively, a "window" exists when the spectrometer is operated differentially. Electronic discrimination of energies below and above desired levels can be obtained with the differential spectrometer. To achieve this discrimination of observed radiation, complex electronic equipment is re-

quired. Units capable of performing these duties are available or are under development.

The gamma probe employs an empirical method that depends on the similar geometry of the test system and of the calibration setup, and on the existence of homogeneous medium about the probe that is similar to that of the calibration system. The direct emissions from the radioactive source are not detected by the detector tubes, but rather, the reflected or scattered radiations of lower energy are "seen." Under the conditions of the test; that is, identical geometry for test and calibration, the number of reflected photons reaching the detector tubes are inversely proportional to the density of the surrounding medium. The mass absorption coefficients of the elements of low atomic weights are similar. As a result, in systems of soil or sediment, and of water, the overall mass absorption coefficients of the constituents are so similar that the linear absorption coefficient may be substituted. The observed attenuation is then due to changes in density of the medium, as the average path of travel of the photons is unaffected by the small change in number of atoms per unit volume.

On strictly theoretical considerations, a calculation of the density of a medium based on the attenuation of reflected and scattered gamma photons is almost impossible. However, because of factors considered above, it is practical to determine the density of materials that approximate in makeup those for which calibration curves have been determined.

Measurements made by the dual probe can be based on theoretical considerations. Two things are required for this. First, the medium through which the photons will travel, and will be attenuated thereby, must be homogeneous. Second, the detection system must be capable of measuring the emitted monoenergetic gamma photons and only those monoenergetic gamma photons. This can be accomplished by combining a scintillation detector and a discrimination circuit in the electronic recording system. A gamma scintillation differential spectrometer would do just this. The Troxler 200B scaler as used was not capable of full discrimination. Partial discrimination was achieved, however, by adjusting the high voltage and amplifier controls. In effect, all pulses of less energy than the peak cesium-137 photons of 0.66 Mev. (million electron volts) were discriminated. This means that any scattered or reflected cesium-137 photons were not counted as they would be of reduced energy. Hence, the reduction in count rate between a standard and the test medium would be due to the differential attenuation of the emitted gamma photons.

Experimental Results

Laboratory

The procedure for calibration of the gamma probe for determining sediment densities has been described (?). A typical calibration curve is shown in figure 3. This particular curve was

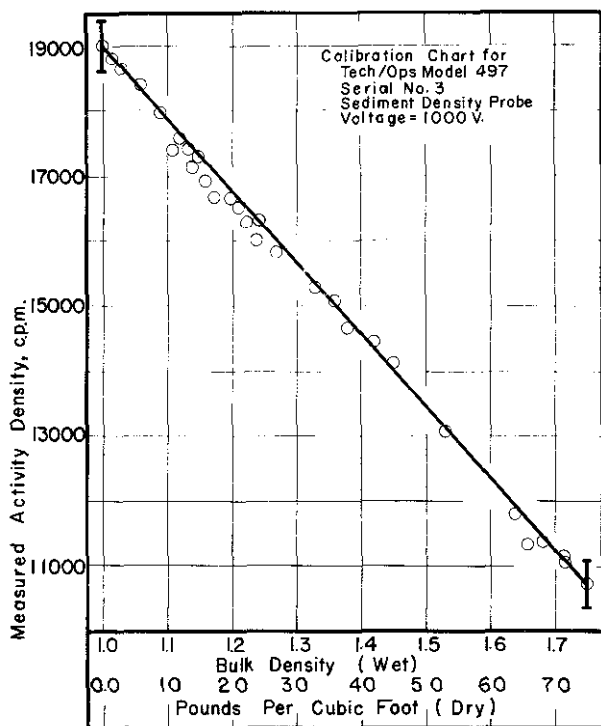


FIGURE 3.—A calibration curve for a single (gamma) probe. The measured activity density is plotted as a function of the density of the prepared sediment-water system. The conversion of density to pounds per cubic foot assumes a specific gravity of 2.65.

valid for only one gamma probe, Technical Operations, Inc., Serial No. 3, for a given period of time. Each gamma probe has a geometry peculiar to it, so each requires a calibration. The calibration curves obtained (?) are parallel, differing only in the magnitude of the measured activity density at a given sediment density. When a new Geiger-Mueller detection tube is installed in the probe or a transistor is replaced in the preamplifier circuit, a check on the applicability of the calibration curve is necessary. Usually, the curve will have shifted slightly from the previous level.

Note that in the calibration curve (fig. 3), the second, or lower, abscissa scale is applicable for sediments whose specific gravity approxi-

mates 2.65. Most sediments encountered in the Mississippi Valley have specific gravities approaching 2.65, that is, of silica or aluminosilicates.

The dual probe was calibrated according to directions of the manufacturer. A magnesium bar and water were used for this purpose. A reading was taken with the magnesium bar interposed between the source and the detection crystal. Then, a measurement was made with water occupying the intervening space. The density of the magnesium bar is 1.75. The measured activity density obtained with the magnesium bar was taken as 100 percent. The measured activity density of the water, specific gravity=1.00, was calculated as a percentage (350 percent) of the magnesium bar activity reading. A calibration curve was constructed from these two primary points. The resulting equation was:

$$\begin{aligned} \text{Sediment density} &= 4.4945 \\ &- 1.3723x \text{ (log of the} \\ &\quad \text{measured activity} \\ &\quad \text{density in counts} \\ &\quad \text{per minute).} \end{aligned} \quad (2)$$

This calibration curve was applicable, as was that for the gamma probe, as long as the electronic components and geometry remained constant in the test system.

The utility of the calibration curve is shown in figure 4, where the measured sediment den-

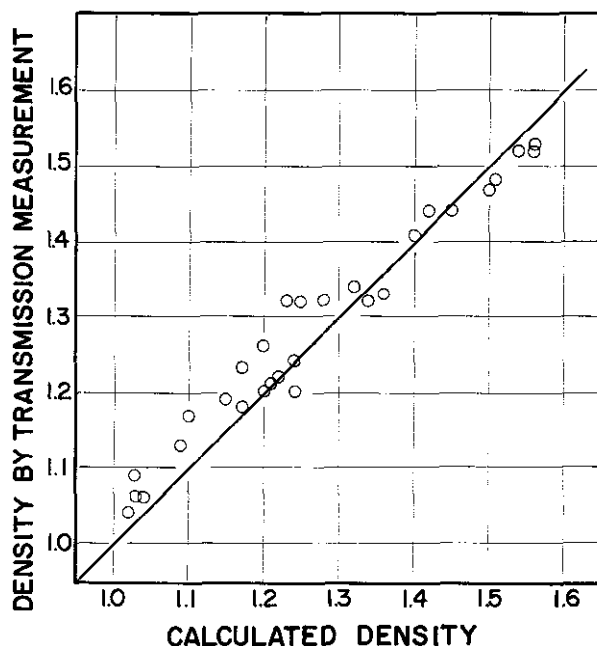


FIGURE 4.—The density of a prepared sediment measured by the dual-probe system plotted as a function of the calculated density. (Specified gravity assumed as 2.65.)

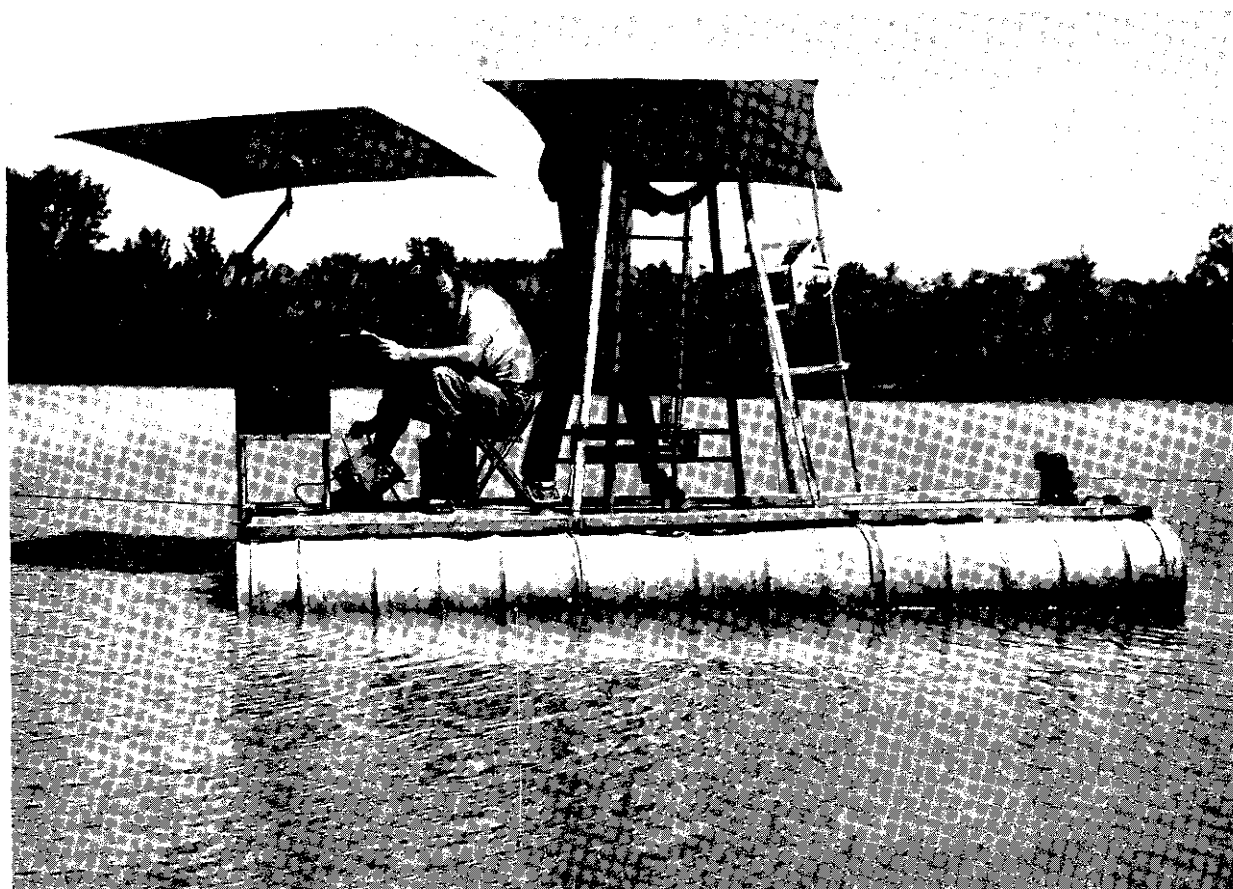


FIGURE 5.—Measuring the density of reservoir sediments. A dual probe, containing a radioactive source and a detection system, has been lowered from the raft into the sediment, and an activity density measurement is being made.

sity values are plotted against calculated sediment densities. Known weights of water and soil materials were mixed in containers, the volume measured, and the density computed. Then the dual probe was placed in the sediment and measurements of density made.

The fit of the observed points to the 1:1 ratio line is good. Many of the points that are somewhat off the line are values for those mixtures that contained considerable amounts of sand in systems of rather low viscosity. The amount of sand in the material employed as sediment was appreciable. The settling velocity of the coarse sand was sufficiently great in suspensions of low viscosity to affect the measurements of density. The data reported in figure 4 represent averages for 5-minute counting times. With careful control of the above-mentioned conditions, the observed densities very closely fit the 1:1 ratio line. To this end bentonite was added to increase the

viscosity where possible and, if not, several shorter counting times were employed.

Field Studies

The gamma probe has been used in sedimentation surveys conducted by the Agricultural Research Service (3, 7). In Mississippi, a small reservoir near Oxford, called Power Line Reservoir, was surveyed in 1959 and 1960 and the gamma probe was used to measure sediment density. The Power Line Reservoir was used again in 1962 in a study to evaluate the utility of the dual probe for field surveys and to compare its performance with that of the gamma probe. All established ranges were resurveyed for sediment density, using both probes.

A specially constructed raft (fig. 5) was used in this survey. This raft is designed so that it may be quickly disassembled for transportation. Its utility for this purpose has been proved by use on a number of reservoirs of varying size and on the Gulf of Mexico.

The raft provides a stable platform for work; an overhead A-frame for the support of the probes and for the raising and lowering of the probes by means of the attached reel and cable; suitable facilities for using the nuclear equipment; and space for storing equipment when not in use. The center-well arrangement permits maximum accessibility to the probes by the crew. The umbrella provides shade for the electronic equipment, which is desirable both as protection against excessive heat and as an aid in reading the dekatron glow tubes of the scaler. On small reservoirs a cable is stretched across the designated range and the raft is attached to this cable and moved to the desired sample location. The distance moved is recorded on the productimeter attached to the raft and through which the cable passes. On larger bodies of water locating the raft is usually done by triangulation either from the raft or from a shore-based member of the field party.

The techniques employed in making density measurements with the gamma probe have been described (3, 7). The techniques used with the dual probe were similar. The measurements of radiation activity were 1 minute in length. The absolute counting rate ranged from 6,000 to 30,000 counts per minute, yielding a standard deviation (σ) of 1.3 to 0.6 percent.

A comparison of typical data obtained by the two probes is shown in figure 6. Here the measured wet density is plotted as a function of sediment depth. As stated previously, the vertical height of the sediment layer measured, or "seen," by the dual probe is less than

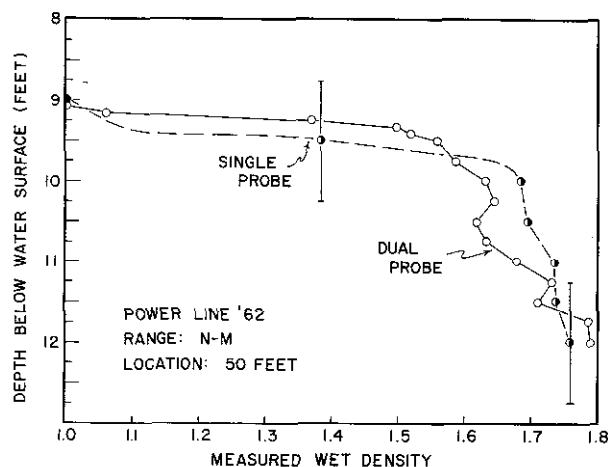


FIGURE 6.—A depth-density profile for a sediment as measured by the gamma and dual probes: Range N-M, 50 feet, Power Line Reservoir, Lafayette County, Miss.

1 inch. Likewise, the vertical resolution of the gamma probe is about 18 inches (7). The diameters of the circles used to indicate the dual-probe test points are approximately scaled to be the vertical dimension measured by the dual probe.

The values for the density of sediments in situ obtained with the two probes are in general agreement. Allowances must be made for the integrating effect of the gamma probe as opposed to the specific density measure provided by the dual probe.

There is some indication of a semisolid sediment, or highly viscous suspension, overlying the consolidated sediment. In figure 6 the dual probe indicates that water exists at a depth of 9 feet 1 inch with sediments of densities of 1.059, 1.371, and 1.501 at 1-, 2-, and 3-inch lower depths. Considerable variation in the density of sediment with an increasing sediment depth was also noted. Typical depth-density profiles are shown in figures 7 to 10. In

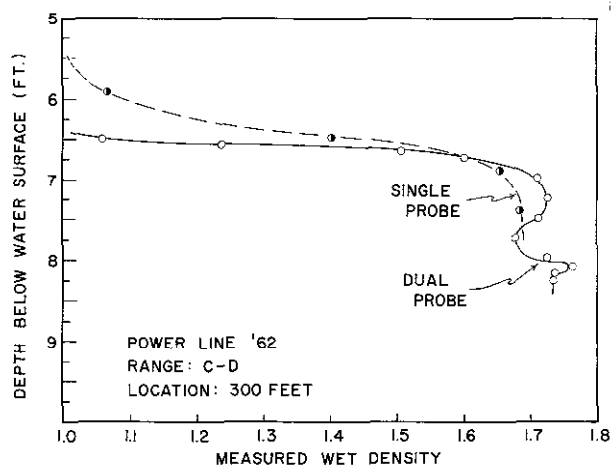


FIGURE 7.—A depth-density profile of a sediment as measured by a gamma and a dual probe: Range C-D, 300 feet, Power Line Reservoir, Lafayette County, Miss.

three of the four profiles the dual-probe density values are shown at greater depths than occur for the gamma probe. It is necessary to recall that the effective length of the gamma probe is 18 inches so that the bottom of the gamma probe is 9 inches below the illustrated, mid-point value of the gamma probe depth. In the case of the dual probe, the two access tubes are lowered as far as possible into the sediments. The lowest recorded dual probe value represents that of the source and detection tube at the bottom of the access tube.

In figure 7 the greater sediment densities at

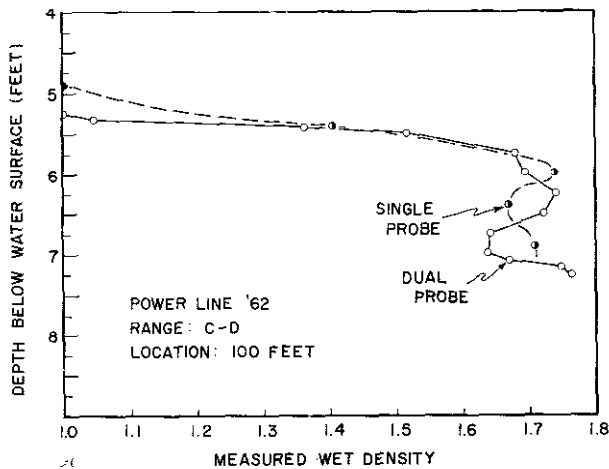


FIGURE 8.—A depth-density profile of a sediment measured with a gamma and a dual probe: Range C-D, 100 feet, Power Line Reservoir, Lafayette County, Miss.

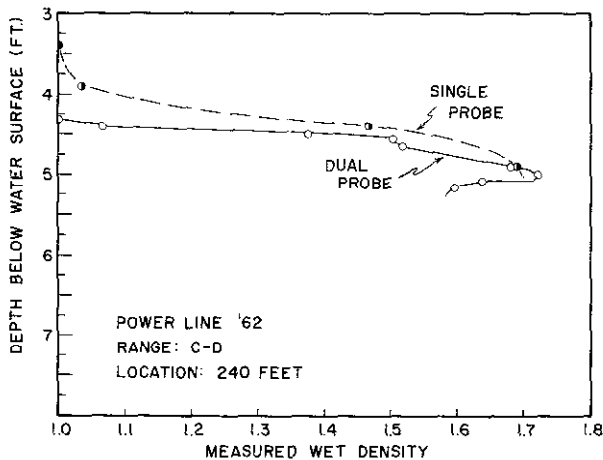


FIGURE 9.—A depth-density profile of a sediment measured with a gamma and a dual probe: Range C-D, 240 feet, Power Line Reservoir, Lafayette County, Miss.

7 and 8 feet are not observed with the gamma probe. The lowest observed gamma probe value, 7 feet 5 inches, represents an integration of the sediment densities from 6 feet 8 inches to 8 feet 2 inches. One may arbitrarily assume that 60 percent of the observed attenuation of radiation occurs with ± 3 inches of the center of the gamma probe; 30 percent with the second 3-inch layer; and that 10 percent occurs in the third 3-inch section. If these assumptions are applied to the data in figures 7 to 10, the results summarized in table 1 are obtained.

The agreement between the calculated and the observed integrated density values is rea-

TABLE 1.—A comparison of selected sediment densities obtained with a gamma probe with those calculated from dual-probe data

Figure	Range	Sample		Wet density	
		Location	Depth	Measured with gamma probe	Calculated from dual-probe data
7...	C-D	Feet 300	Fi. In. 7 5	1.685	1.700
8...	C-D	100	6 5	1.670	1.707
8...	C-D	100	6 11	1.708	1.685
9...	C-D	240	4 11	1.690	1.741
10...	H-G	190	9 11	1.574	1.529

sonably good with the exception of the data taken from the 240-ft. profile in range C-D (fig. 9). In the latter there are insufficient dual-probe values to estimate adequately the density in an 18-inch vertical zone.

In three depth-density profiles (figs. 7, 8, and 9), the dual-probe data indicated variations in the density of the deposited sediments. In figure 8, the gamma probe also indicates this fact, although the influence of the integrating characteristic of the gamma probe reverses the true location of high and low density layers. Similar agreement in the measured density values by the two probes continued for other locations in range C-D in addition to those illustrated. At times, however, agreement was less good: In figure 10 considerable discrepancy is noted at the lower depths. This discrepancy and those in other data not shown may be due to any one or more of the following reasons. The location of the test spot is subject to variation. Lateral drift of the raft may occur due to wind currents. Even on quiet days, some

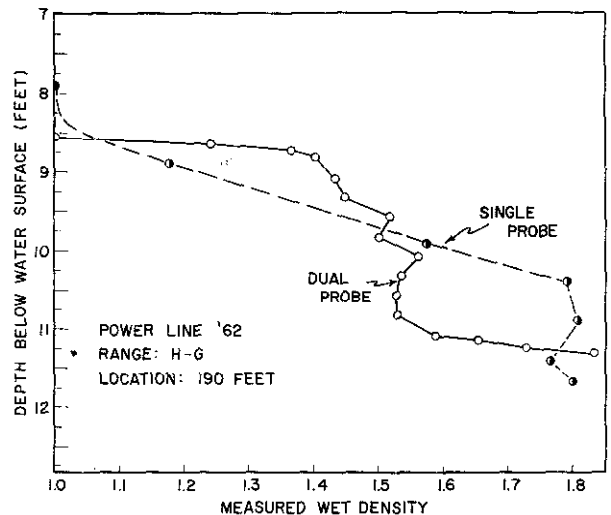


FIGURE 10.—A depth-density profile of a sediment measured with a gamma and a dual probe: Range H-G, 190 feet, Power Line Reservoir, Lafayette County, Miss.

yaw of the raft occurs. The measurement of distance traveled on the cable is subject to error because slippage between the cable and productimeter may occur. Corrections for changes in the water level elevations were made. It is difficult, however, to measure the water level at the probe within an inch or two. The surveys with the two probes were made on different days, so relocation was subject to error. Although care was taken to insure that the probes were inserted vertically into the sediment, this was not always accomplished because of movements of the raft in the water. The data reported are assumed for duplicate determinations, but the actual points measured may be somewhat separated.

To summarize the representative data obtained with the dual probe, the sediment densities measured for the entire cross section C-D are shown in figure 11. The existence of a few inches of semiconsolidated sediment is seen, particularly between 140 and 180 feet. The high density sediments, those over 70 pounds dry weight per cubic foot, are confined to the edges of the larger channel and at the bottoms of the two smaller channels. Some penetration of the dual probe into the original ground is observed in the deep part of the illustrated cross section. Otherwise, little or no penetration of this material was achieved with this dual probe.

The existence of layers of sediment of different densities is readily apparent on examination of the data shown in figure 11. A number of factors have contributed to this layering, but discussion of them is beyond the scope of this paper. It was observed, however, in the course of the survey that those areas where densities were higher usually contained larger amounts of sand. The lack of high density material in the deeper parts of the cross section could, therefore, be ascribed to the settling out of the sand before the sediments were carried into the deeper pool areas.

Discussion

The performance of the dual probe has been generally satisfactory. In the laboratory, density measurements were made with the same precision and reliability as with the gamma probe. In the field, the performance of the dual probe was less dependable than was that of the gamma probe. This is due in part to the electronic circuitry comprising the scintillation detection system. Geiger-Mueller detector tubes, as employed in the gamma probe, function similarly over a range of applied voltages. Once this voltage plateau is reached, small changes in the applied voltage have little effect on the performance or sensitivity of the system.

On the other hand, the sensitivity of a scintillation system to radiation is directly proportional to the applied voltage—the greater the voltage, the greater the sensitivity and the more counts that are observed. In order, therefore, for a scintillation system to operate reliably, it is necessary to have rigid control of the high voltage supply. As presently constructed, the Troxler Model 200B does not provide adequate high voltage control for routine measurements. In the laboratory, and in the field when used by well-trained persons, the dual probe is presently adequate. For routine field work of the type where the gamma probe may now be used, the dual probe is inadequate in its present state. However, within the noted limitations, its performance is adequate and its use permits the collection of unique data. As auxiliary portable equipment becomes available, the performance of the dual probe will be improved.

Summary and Conclusions

The use of nuclear techniques for measuring the densities of materials by the attenuation of gamma rays has been successfully applied to reservoir sedimentation studies. The performance of a single (gamma) probe and of a dual-probe (gamma) system for measuring densities of reservoir sediments in situ was evaluated. Both field and laboratory tests were conducted.

Previous work has indicated the gamma probe is unsuitable for measuring the densities of sediments in narrow layers or near phase interfaces. The dual probe provides means whereby such measurements can be made. The reliability and precision of these measurements is equal to that obtained with the gamma probe for larger masses of sediments.

In its present state of development, the dual probe is capable of measuring the density of 1-inch layers of sediment unaffected by the material above or below. Such measurements of density have been made successfully in up to 3 feet of sediment beneath 10 to 15 feet of water.

The performance of the dual probe in the laboratory was entirely satisfactory. However, the field performance of the dual probe in its present state of development is not equal to that of the gamma probe. An increase in the ruggedness of the design and in the reliability of the observed measurements is necessary.

The performance of the gamma probe is such that its use in routine sediment surveys is now possible. Additional design modifications and improved measurement techniques must be devised before the dual probe can be employed in the average sedimentation survey. However, for special purpose surveys, the dual probe can now be employed by suitably trained personnel.

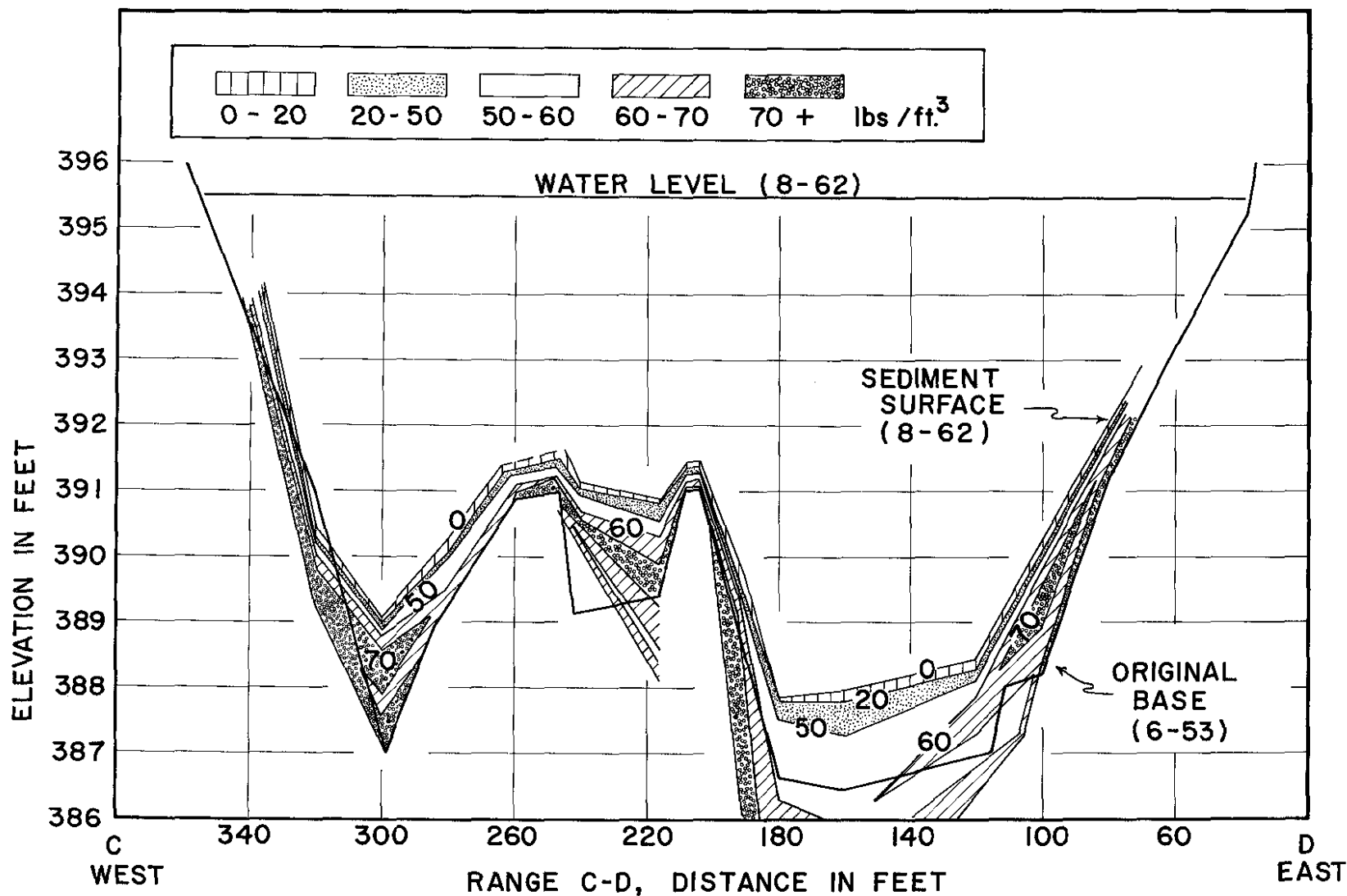


FIGURE 11.— Sediment density, in pounds per cubic foot of dry material, measured by the dual probe: Range C-D, Power Line Reservoir, Lafayette County, Miss. 1962

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SEDIMENT WITHDRAWAL INVESTIGATION — GUERNSEY RESERVOIR

[Paper No. 91]

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Synopsis

Guernsey Reservoir, which was constructed primarily for irrigation storage, was drawn down purposely to produce increased sediment concentration releases. Data were collected for 3 years to determine sediment movement and redistribution of substantial sediment deposits within the reservoir area. Inflow and outflow rates, as well as the drawdown rate, were varied to establish sediment outflow from the reservoir under various conditions. Studies of field data were made to estimate the efficiency and length of life that can be expected from future reservoir drawdowns with respect to removing sediment from the reservoir.

Results indicate that controlled fluctuated reservoir inflow (clear water) does not, in itself, significantly improve the maximum sediment concentration outflow. The relative efficiency of sediment withdrawal by reservoir drawdown has varied considerably with fluctuation of reservoir storage and redistribution of sediment within the reservoir. Sediment withdrawal is greater at low reservoir storage and will be limited if a storage cushion is required at the bottom of a reservoir for power operations. Increased concentrations of sediment withdrawn from the reservoir can be obtained by increases in inflow at minimum storage. It is expected that a stable channel gradient will ultimately prevail through the reservoir and, combined with some continued channel widening, will provide sufficient sediments for with-

drawal if reservoir storage can be decreased below 2,000 acre-feet and inflows can be fluctuated.

Introduction

For many years, the irrigation systems in the North Platte area have experienced heavy intake of sediments with the diverted canal waters. Canal cleaning problems were severe, but canal seepage losses appeared to be less. With the construction of Guernsey Dam and Reservoir in 1927, clear water releases tended to increase the canal seepage losses and canal bank erosion. As sediment accumulation increased in Guernsey Reservoir, normal reservoir operations during the past decade removed sediments from the reservoir and again beneficial aspects appeared in the downstream canals and laterals. With the construction of Glendo Dam and Reservoir in 1957, about 16 miles upstream, sediment inflow to Guernsey Reservoir was radically reduced. With an integrated operation of the North Platte River storages for irrigation and power purposes, it was necessary to evaluate the qualitative and quantitative extent of sediment withdrawals from Guernsey Reservoir, the effectiveness of the sediment-laden water upon canal seepage, and the most effective means of extracting the reservoir sediments—now and in the future.

General Description

Guernsey Dam, on the North Platte River in Wyoming, was completed by the Bureau of

Reclamation in 1927, as a feature of the North Platte Project. The reservoir behind the dam is used for the storage of irrigation water, reregulating upstream storage releases, and for power production. Irrigation releases from Guernsey Reservoir to downstream lands average about 1,130,000 acre-feet per year. The present surface area of the reservoir is about 2,380 acres at crest elevation of 4,420; the existing storage at this elevation is about 44,800 acre-feet, as compared to the original capacity of 73,810 acre-feet. The length of the reservoir is 14.6 miles; the width varies from less than 500 feet to approximately $\frac{1}{4}$ mile for a distance of about 5 miles above the dam. Above

this reach, the reservoir for over a 4-mile reach widens, ranging from $\frac{1}{2}$ mile to almost 1 mile. The upper reach of the reservoir is narrow, ranging from 300 feet to about $\frac{1}{4}$ mile.

Although the North Platte drainage area above the Guernsey Dam is about 16,200 square miles, the sediment contributing area has varied from 5,500 square miles in 1927 (below Pathfinder Dam), 5,400 square miles in 1938 (below Alcova Dam) and to about 700 square miles in 1957 (below Glendo Dam). Up to 1957, the reservoir has been subject to a very high sedimentation rate, losing 39.3 percent of the original capacity. Since construction of Glendo Dam, or since 1957, the estimated rate

TABLE 1.—Sediment concentration data for reservoir drawdown periods, Guernsey Reservoir 1959–62

Date	Above Guernsey		In Guernsey			Below Guernsey Sta III
	Sta I	Sta II	Sta IV	Sta V	Sta VI	
1959	P.p.m.	P.p.m.	P.p.m.	P.p.m.	P.p.m.	P.p.m.
Aug. 6.						114
Aug. 7.		90		4,270	3,750	476
Aug. 8.	18	68				847
Aug. 9.						585
Sept. 16.	8	95				10
1960						
July 2,3,4	11	11				11
July 7.						11
July 9.	14	14				18
July 10.	12	11	600	20	6	33
July 11.		14	1,060	150	25	33
July 12.	11	14	560	730	50	84
July 13.		15	560	5,850	530	310
July 14.		17	375	4,930	950	235
July 15.	11	28	360	2,620	660	310
July 16.	11	14	355	2,900	2,050	169
July 17.		11	330	2,140	4,160	202
July 18.		11	332	2,140	5,970	264
July 19.		11	352	2,140	3,000	255
July 20.						156
July 21.						88
July 22.						90
July 23.						69
July 24.						62
July 25.						50
July 26.						28
July 27.						11
July 28.						17
July 29.						15
Aug. 4.	22	23				18
Aug. 5.						15
Aug. 6.						25
Aug. 7.	19	15				19
Aug. 8.		18	370	25	17	21
Aug. 9.		18	350	70	20	61
Aug. 10.		22	550	160	35	63
Aug. 11.		29	600	240	55	88
Aug. 12.	28	29	620	530	340	123
Aug. 13.		16	470	2,600	570	178
Aug. 14.		23	525	2,190	3,080	369
Aug. 15.		26	560	1,830	1,180	195
Aug. 16.		18	352	1,060	1,260	120
Aug. 17.		23	598	575	1,330	154
Aug. 18.		31	580	1,700	1,575	134
Aug. 19.	30					95
Aug. 20.						57
Aug. 21.						65
Aug. 22.						71
Aug. 23.	21					76
Aug. 24.						82

Date	Above Guernsey		In Guernsey			Below Guernsey Sta III
	Sta I	Sta II	Sta IV	Sta V	Sta VI	
1960	P.p.m.	P.p.m.	P.p.m.	P.p.m.	P.p.m.	P.p.m.
Aug. 31.	19					
Sept. 9.	15					
1961						
July 8.						15
July 18.						14
July 20.		12	300	54	48	
July 22.						39
July 23.		7	400	190	54	40
July 24.		10	180	300	56	34
July 25.				300	74	46
July 26.		13	161	3,900	255	86
July 27.	46	74	840	1,250	4,600	390
July 28.		12	225	1,100	2,400	460
July 29.			330	1,370	2,930	540
July 30.			175	900	1,500	560
July 31.			260	770	335	76
Aug. 1.				900	52	37
Aug. 2.				125	56	34
Aug. 3.			105	74	40	24
Aug. 7.		7	350	29	50	10
Aug. 20.						13
1962						
July 12.	21					
July 21.						27
July 22.						50
July 23.						52
July 24.			61	440	235	
July 25.						44
July 26.		26	39	84	370	40
July 27.			100	210	285	34
July 28.		20	130	720	270	60
July 29.			92	2,900	460	125
July 30.			88	1,700	2,000	200
July 31.			66	1,060	880	190
Aug. 1.			87	1,140	3,750	410
Aug. 2.			115	920	2,050	780
Aug. 3.			103	920	1,900	780
Aug. 4.		20	102	810	1,375	590
Aug. 5.			145	870	1,405	620
Aug. 6.			105	990	1,610	710
Aug. 7.		25	90	890	1,410	690
Aug. 8.			160	1,270	1,720	770
Aug. 9.			94	310	270	115
Aug. 10.			70	79	365	65
Aug. 11.			35	31	52	32
Aug. 12.		28	29	81	56	22
Aug. 13.						15
Aug. 21.						18
Aug. 22.						15
Aug. 31.						14

of sediment inflow is approximately 150 acre-feet per year as compared to the historic sediment accumulation of 970 acre-feet per year.

Available Data

Under the Bureau of Reclamation's Soil and Moisture Conservation Program, data pertaining to sediment concentration of reservoir inflow, sediment movement within the reservoir

proper, and sediment concentration of reservoir outflow were collected relative to the Guernsey Reservoir drawdown periods 1960, 1961, and 1962. These data are summarized in table 1. Data relative to the reservoir drawdown periods are tabulated in table 2. A general graph of reservoir operations from 1957 to 1962 and pertinent sediment concentration data collected during the withdrawal periods

TABLE 2.—Guernsey Reservoir drawdown data, 1959–62

Date (1)	Water surface elev. (2)	Storage (3)	Inflow (4)	Outflow (5)	Difference, column 4–5 (6)	Maximum concentration inflow (7)	Concentration outflow (8)	Trap efficiency, (8) ÷ (7) (9)
	Feet	Acro-feet	C.f.s.	C.f.s.	C.f.s.	P.p.m.	P.p.m.	Percent
1959								
Aug. 7, 1959.....	4,376.6	2,600	4,948	5,190	-242	4,270	476	11.1
1960								
July 10.....	4,405.6	16,900	2,916	4,960	-2,044	600	33	5.5
July 11.....	4,399.2	10,300	2,886	5,000	-2,114	1,060	33	3.1
July 12.....	4,392.2	5,500	2,868	5,000	-2,132	730	84	11.5
July 13.....	4,386.6	3,200	3,975	4,980	-1,005	5,850	310	5.8
July 14.....	4,383.0	2,200	4,424	4,980	-556	4,930	235	4.8
July 15.....	4,384.9	2,700	5,155	4,980	+175	2,620	310	11.8
July 16.....	4,383.4	2,300	4,800	5,020	-220	2,900	169	5.8
July 17.....	4,381.4	1,750	4,796	5,040	-244	4,160	202	4.8
July 18.....	4,380.9	1,650	4,987	5,040	-53	5,970	264	4.4
July 19.....	4,389.1	4,150	6,105	5,080	+1,025	3,000	255	8.5
Aug. 8.....	4,404.98	16,000	3,981	4,800	-819	370	21	5.7
Aug. 9.....	4,403.29	14,100	3,705	4,820	-1,115	350	61	17.4
Aug. 10.....	4,401.19	12,000	3,616	4,840	-1,224	550	63	11.4
Aug. 11.....	4,398.25	9,500	3,423	4,880	-1,475	600	88	14.7
Aug. 12.....	4,394.00	6,600	3,676	4,820	-1,144	620	123	19.8
Aug. 13.....	4,388.3	3,800	3,613	4,710	-1,097	2,600	178	6.8
Aug. 14.....	4,381.0	1,650	4,404	4,630	-226	3,080	369	12.0
Aug. 15.....	4,381.5	1,750	4,361	4,480	-119	1,830	195	10.6
Aug. 16.....	4,382.2	1,950	4,224	4,360	-136	1,260	120	9.5
Aug. 17.....	4,380.4	1,550	4,194	4,350	-156	1,330	154	11.6
Aug. 18.....	4,381.1	1,700	5,850	4,270	+1,580	1,700	134	7.9
1961								
July 23.....	4,400.7	11,600	3,979	4,570	-591	400	40	10.0
July 24.....	4,399.2	10,300	3,662	4,570	-908	300	34	11.3
July 25.....	4,396.35	8,050	3,010	4,590	-1,580	300	46	15.3
July 26.....	4,389.40	4,250	3,136	4,610	-1,474	3,900	86	2.2
July 27.....	4,380.10	1,500	4,258	4,590	-332	4,600	390	8.5
July 28.....	4,378.90	1,200	4,565	4,610	-45	2,400	460	19.2
July 29.....	4,378.00	1,050	3,902	3,960	-58	2,930	540	18.4
July 30.....	4,378.30	1,100	4,534	4,160	+374	1,500	560	37.3
July 31.....	4,384.00	2,450	2,933	2,340	+593	770	76	9.9
Aug. 1.....	4,388.40	3,900	2,347	1,750	+597	900	37	4.1
Aug. 2.....	4,392.90	5,900	2,446	1,280	+1,166	125	34	27.2
Aug. 3.....	4,398.30	9,500	3,797	1,030	+2,767	105	24	22.9
Aug. 7.....	4,407.1	19,000	4,058	4,230	-172	350	10	2.9
1962								
July 26.....	4,404.77	16,800	3,596	4,720	-1,124	370	40	10.8
July 27.....	4,402.95	14,600	2,846	4,631	-1,785	285	34	11.9
July 28.....	4,398.0	9,800	3,049	4,479	-1,430	720	60	8.3
July 29.....	4,390.8	4,800	3,230	4,422	-1,192	2,900	125	4.3
July 30.....	4,388.2	3,550	3,928	4,270	-342	2,000	200	10.0
July 31.....	4,385.6	2,500	3,208	3,791	-583	1,060	190	17.9
Aug. 1.....	4,380.1	1,500	3,348	3,740	-392	3,750	410	10.9
Aug. 2.....	4,377.9	1,150	3,442	3,536	-94	2,050	780	38.0
Aug. 3.....	4,377.1	1,000	3,555	3,519	+36	1,900	780	41.0
Aug. 4.....	4,377.9	1,150	3,759	3,723	+36	1,375	590	42.9
Aug. 5.....	4,378.0	1,150	4,241	4,090	+151	1,405	620	44.1
Aug. 6.....	4,379.6	1,400	4,858	4,840	+18	1,610	710	44.1
Aug. 7.....	4,379.2	1,350	4,758	4,726	+32	1,410	690	48.9
Aug. 8.....	4,380.7	1,600	6,089	4,490	+1,599	1,720	770	44.8
Aug. 9.....	4,395.4	7,800	7,176	4,669	+2,507	310	115	37.1
Aug. 10.....	4,401.8	13,300	7,007	4,783	+2,224	365	65	17.8
Aug. 11.....	4,406.9	19,800	6,772	4,840	+1,932	52	32	61.5
Aug. 12.....	4,407.7	20,700	5,246	4,745	+501	81	22	27.2

are shown in figure 1. Included in the investigations during the drawdown periods were information on bank erosion of the exposed sediment deposits, channel cross sections in the reservoir area, water surface slopes, and a resurvey of ranges in the lower end of the reservoir. The 1957 resurvey of Guernsey Reservoir was used in obtaining prior data on reservoir sediment ranges, size analyses, and densities of reservoir sediment deposits.¹

Sediment Characteristics

Suspended sediment samples were obtained within the reservoir study reach and from the reservoir releases during the withdrawal investigations. The average size analyses curves for these respective locations are shown in figure 2. Sampling of the reservoir sediment deposits was also performed during these periods. Samples were taken of the channel bed material and the quasi overbank deposit within the reservoir area (fig. 2).

Unit weight determinations were based on samples of reservoir sediment deposits, gamma probe investigations of in situ sediment deposits, and computations of volume weights relative to the size analyses of suspended sediments. An average unit weight of 67 pounds per cubic foot was determined from these methods and was employed in the analysis of the sediment withdrawal data.²

Reservoir Sediment Distribution

Resurvey of the reservoir in 1957 determined a sediment accumulation of 29,100 acre-feet in the reservoir proper. Based on inflow-outflow sediment concentration data collected during the period 1957-1962, an estimated 117 acre-feet of sediment has been removed since the 1957 survey. Of the 117 acre-feet total sediment removed, 88 acre-feet were removed during the withdrawal periods 1960, 1961, and 1962, or approximately 75 percent of the total removed. The longitudinal distribution of sediment deposits determined at the various reservoir resurveys and investigations is shown in figure 3. The thickness of the sediment depth has ranged up to almost 40 feet. The thalweg profile depicts the slope of the deposits at various locations in the reservoir area and their change relative to time. The slope of the delta topset bed is estimated to be about 2.5 feet per mile as compared to 5.7 feet per mile for the original river channel slope.

Comparing the 1957 profile with subsequent

profiles (fig. 3) shows a decrease in the upper reservoir sediment deposits and an increase in sediment deposition in the lower portion of the reservoir. Below elevation 4,386 feet, there was a reduction in reservoir capacity of about 31 percent. A typical cross section of sediment deposits in the lower reservoir reach is shown in figure 4. The redistribution of sediment deposits during the 5-year interim period occurred relative to drawdown periods, at which time the main channel traversing the reservoir area scoured the upper sediment beds and conveyed the bulk of the scoured sediment to the lower portion of the reservoir. This movement, however, applies in general to the thalweg, as defined by the single profile shown for respective years.

Sediment Withdrawals

The aforementioned data, tables 1 and 2, were analyzed to determine possible correlations, if any, between inflow-outflow rates, drawdown rates, or storage fluctuation on the sediment concentration of the reservoir discharge. The parameter used for determining the effect of the drawdown operation will be referred to as relative efficiency, being defined as the ratio of the observed suspended sediment concentration of the reservoir releases to the maximum observed suspended sediment concentration inflow to the reservoir. The maximum observed inflow concentrations to the reservoir occurred in proximity to the reservoir estuary, which fluctuated during and between drawdown periods.

Effect of Inflow-Outflow Rates

During the drawdown investigations, the reservoir inflow ranged between approximately 2,000 and 7,000 c.f.s. with controlled releases from the upstream Glendo Reservoir. The respective inflow-outflow discharges for the drawdown periods 1960-62 are tabulated in table 2 and are shown graphically in figures 5, 6, and 7. The efficiency (col. 9, table 2) appears to be affected by inflow-outflow fluctuations; however, there appears to be no significant trend. The relative efficiency of the sediment withdrawal is somewhat higher when the inflow exceeds the outflow; however, this does not give an entirely independent parameter. The effect of increased inflow, coupled with minimum reservoir storage, does appear to have impact on relative efficiency and absolute maximum sediment inflow and outflow. These effects are to be discussed later in this paper.

Effect of Drawdown Rate

The drawdown rate in acre-feet per day was determined from the respective slopes of the drawdown storage curves shown in figures 5, 6, and 7. The storage curves are based on revised

¹ BUREAU OF RECLAMATION. SEDIMENTATION RESURVEY OF GUERNSEY RESERVOIR, 1957, NORTH PLATTE PROJECT, WYOMING AND NEBRASKA. Sedimentation Section, Hydrology Branch. August 1958.

² See paper 82 of this Conference.

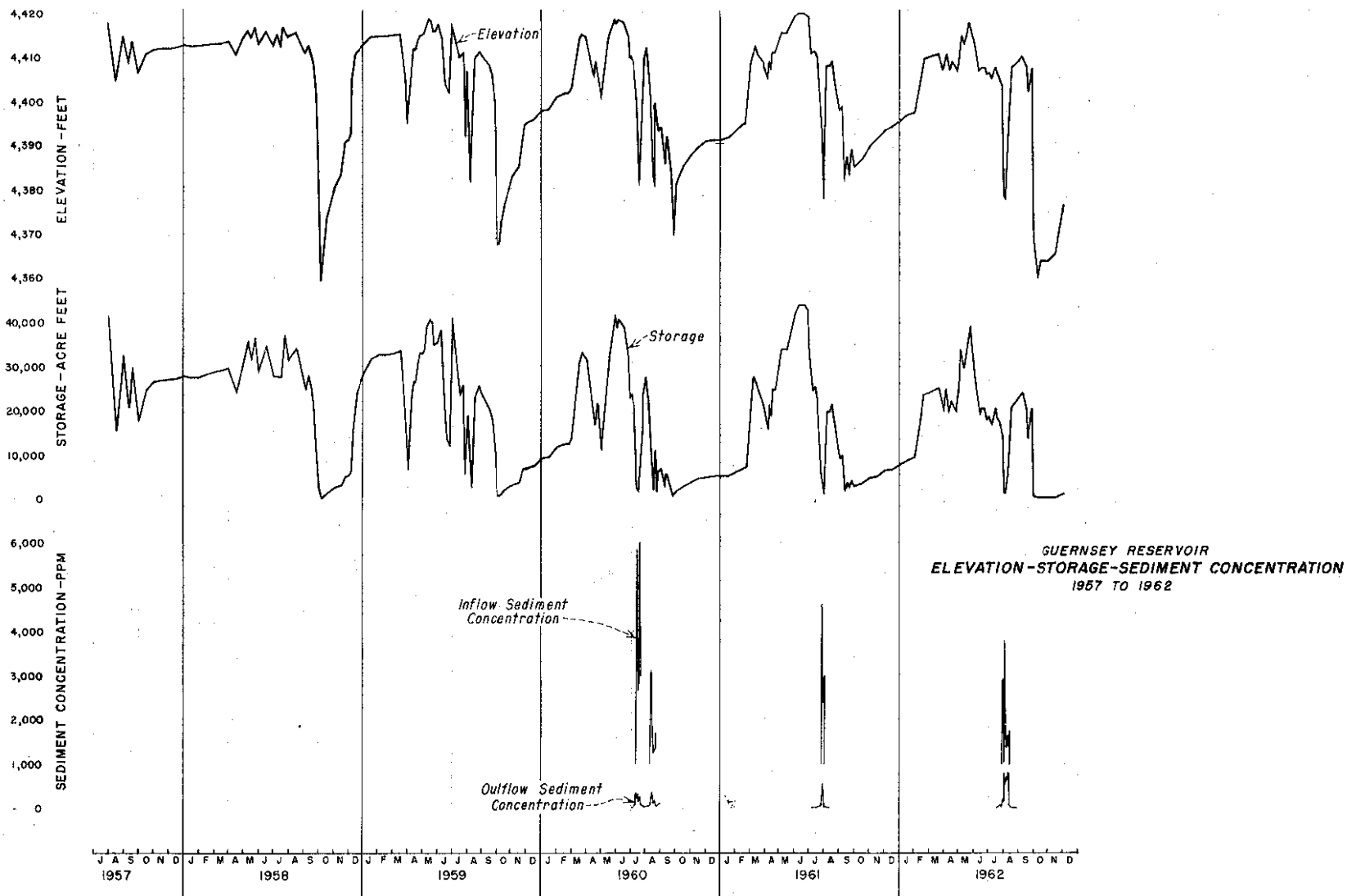


FIGURE 1. — Elevation-storage-sediment concentration, 1957 to 1962.

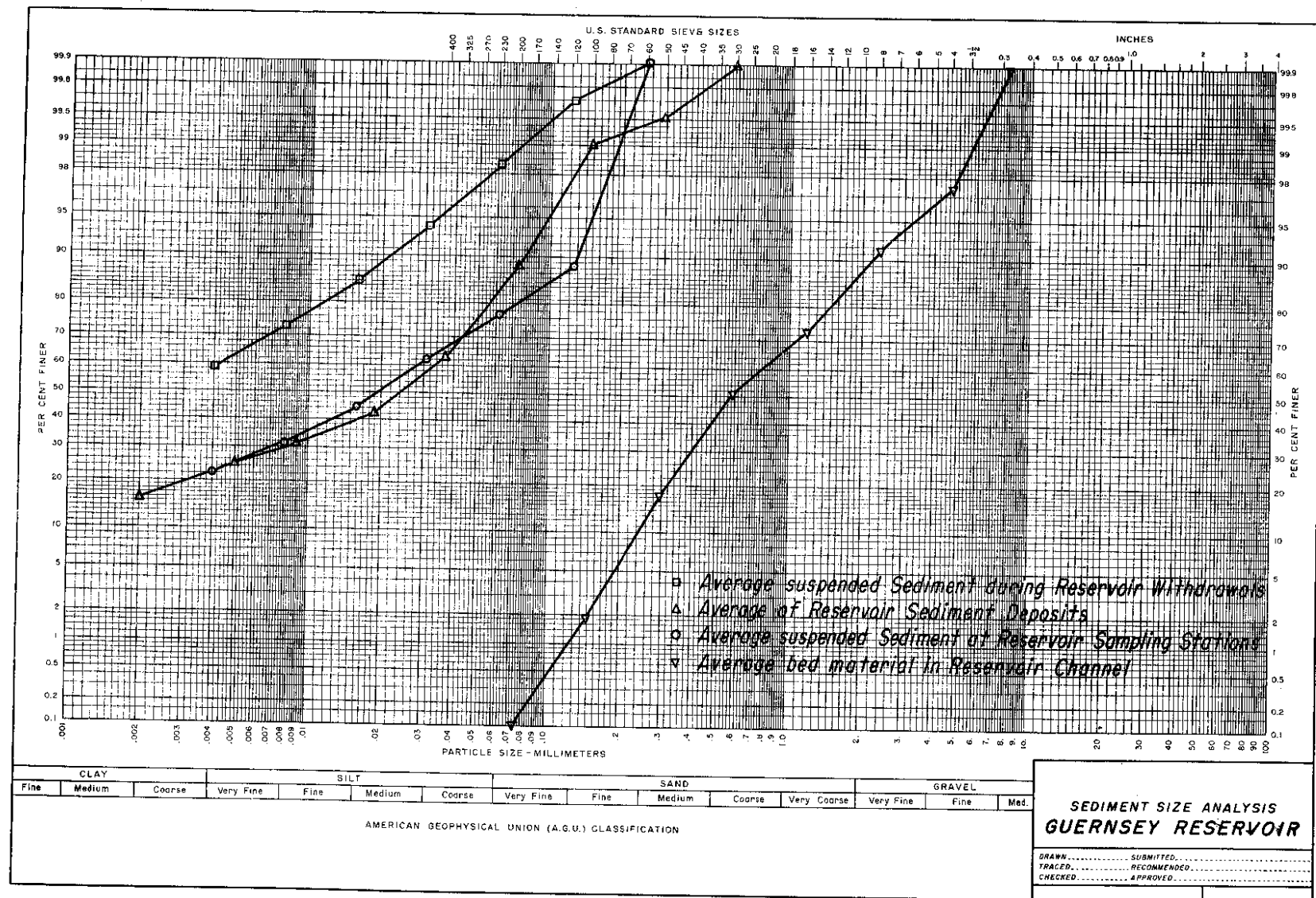


FIGURE 2. — Sediment size analysis.

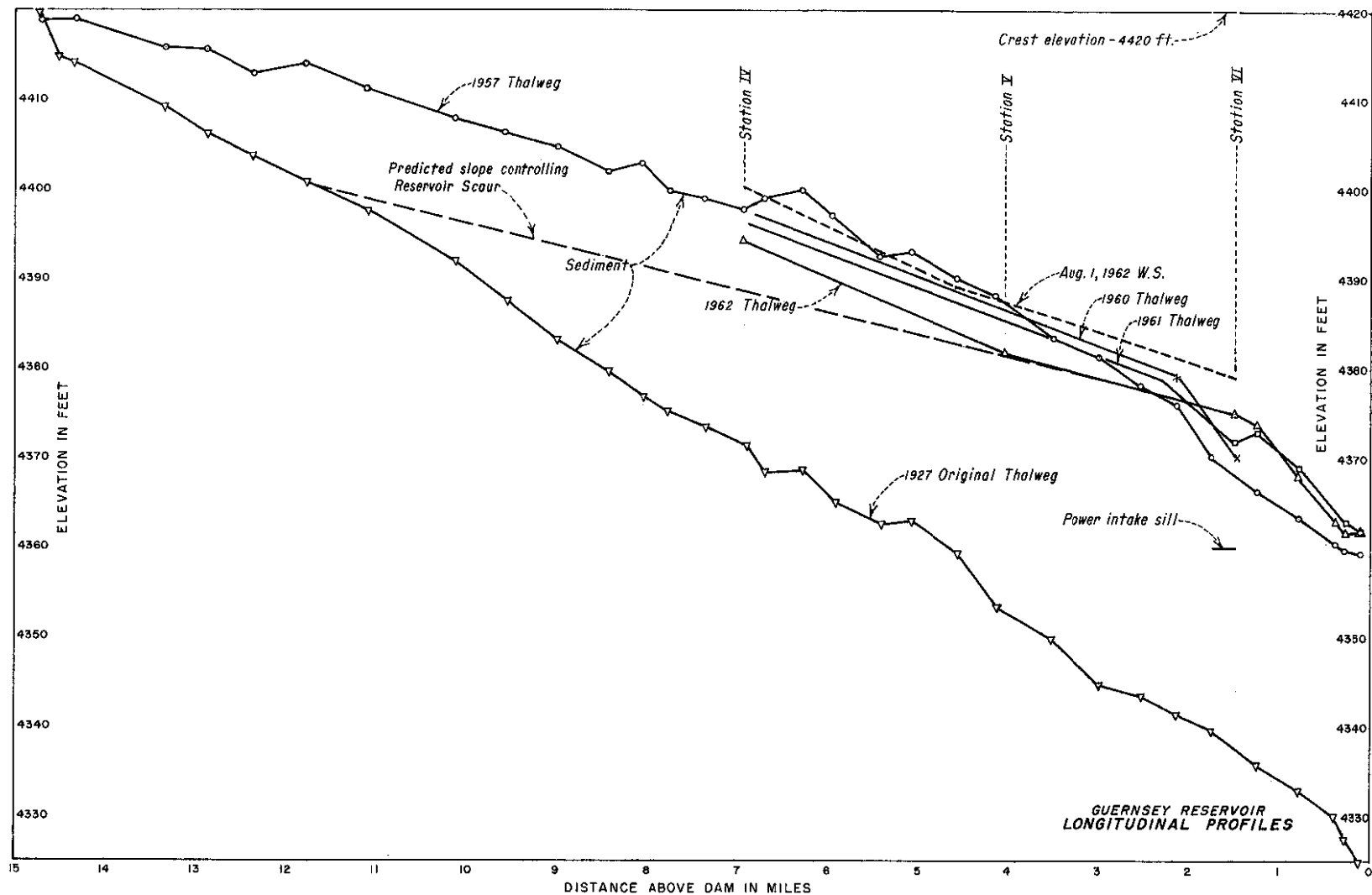


FIGURE 3. — Longitudinal profiles of sediment deposits.

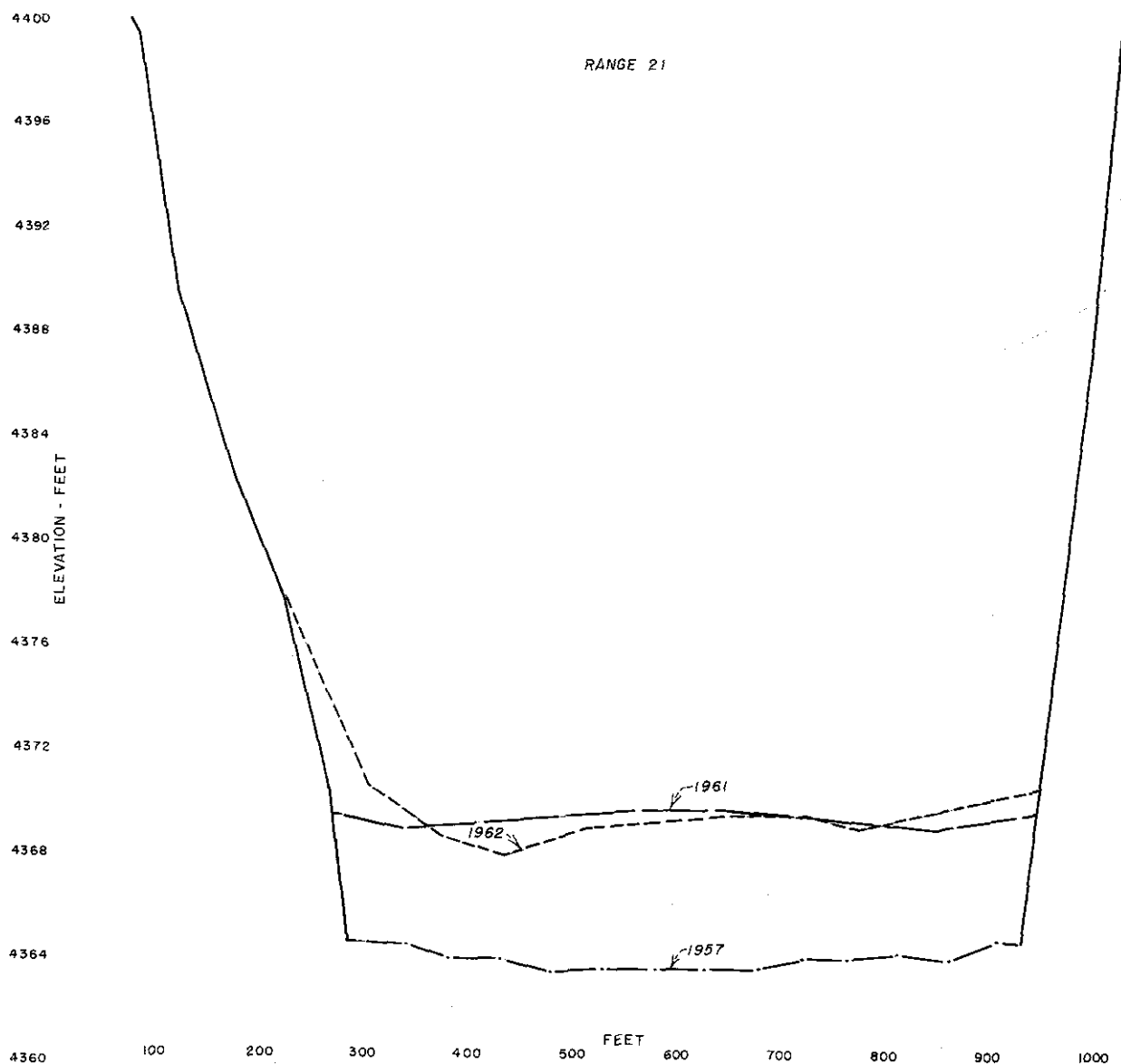


FIGURE 4. — Typical cross section of sediment deposits in lower reservoir.

capacity curves prepared from resurvey data pertaining to the sediment ranges in the reservoir study reach. The rate of drawdown had no apparent effect upon sediment concentrations in the reservoir releases, as indicated in table 3.

Effect of Storage Fluctuation

The effect of storage on the sediment concentration of reservoir releases is shown in figures 5, 6, and 7. As previously pointed out, the rates of storage fluctuation do not have significant effect on the relative efficiency; however, the period of "minimum storage" appears to be relative to increased efficiency and

is certainly indicative of maximum sediment inflow and outflow. The correlation of storage to sediment concentrations of reservoir releases is shown in figure 8. A differentiation was made between sediment concentration obtained during increasing and decreasing storage. The dispersion of sediment concentration relative to increasing and decreasing storage gave no distinguishing trends in this respect. A curve was drawn representing the trend of the storage concentration relation of the plotted data. Outflow sediment concentrations relative to storage below approximately 4,000 acre-feet can have a

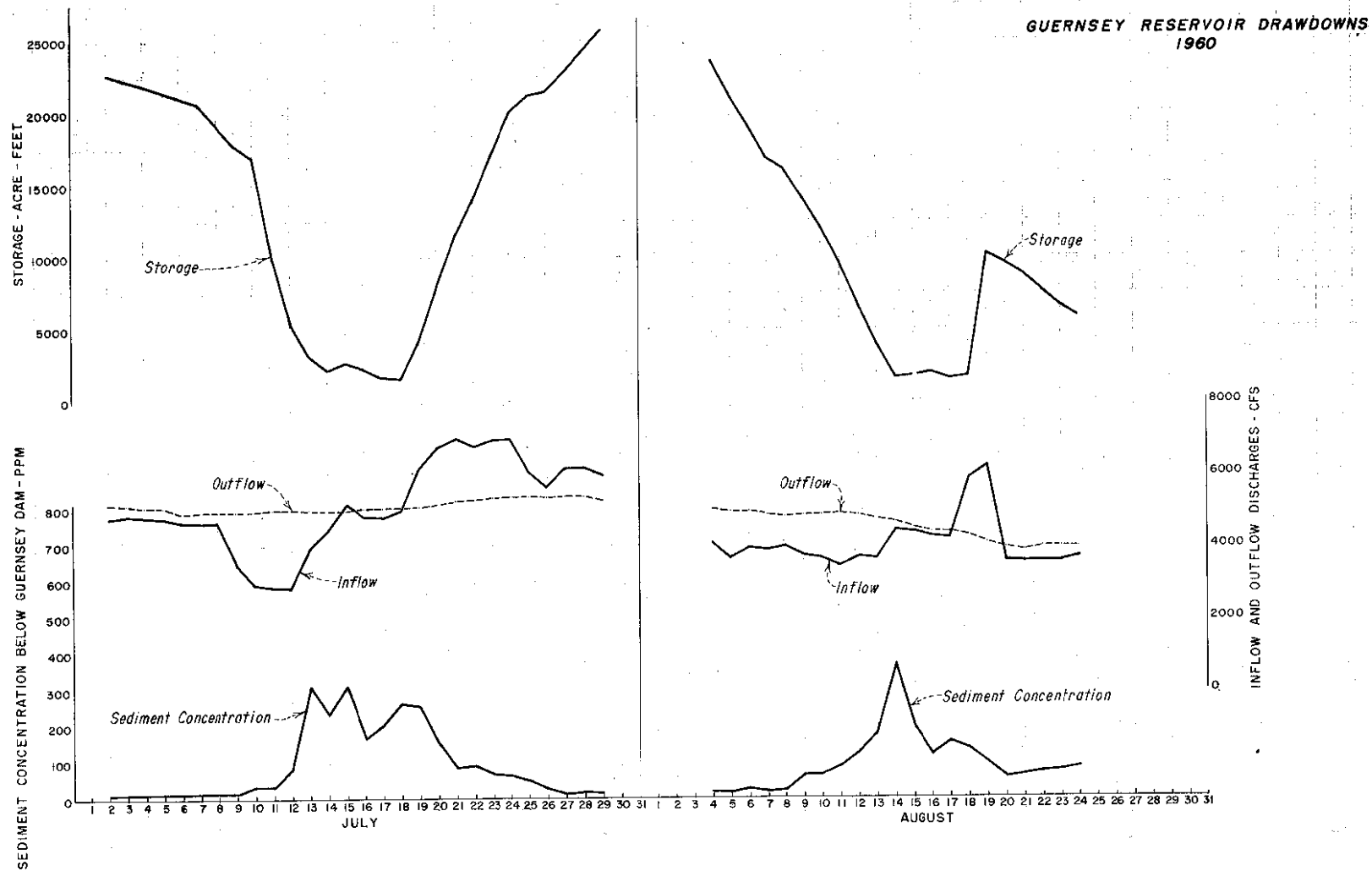


FIGURE 5. — Guernsey Reservoir drawdowns, 1960.

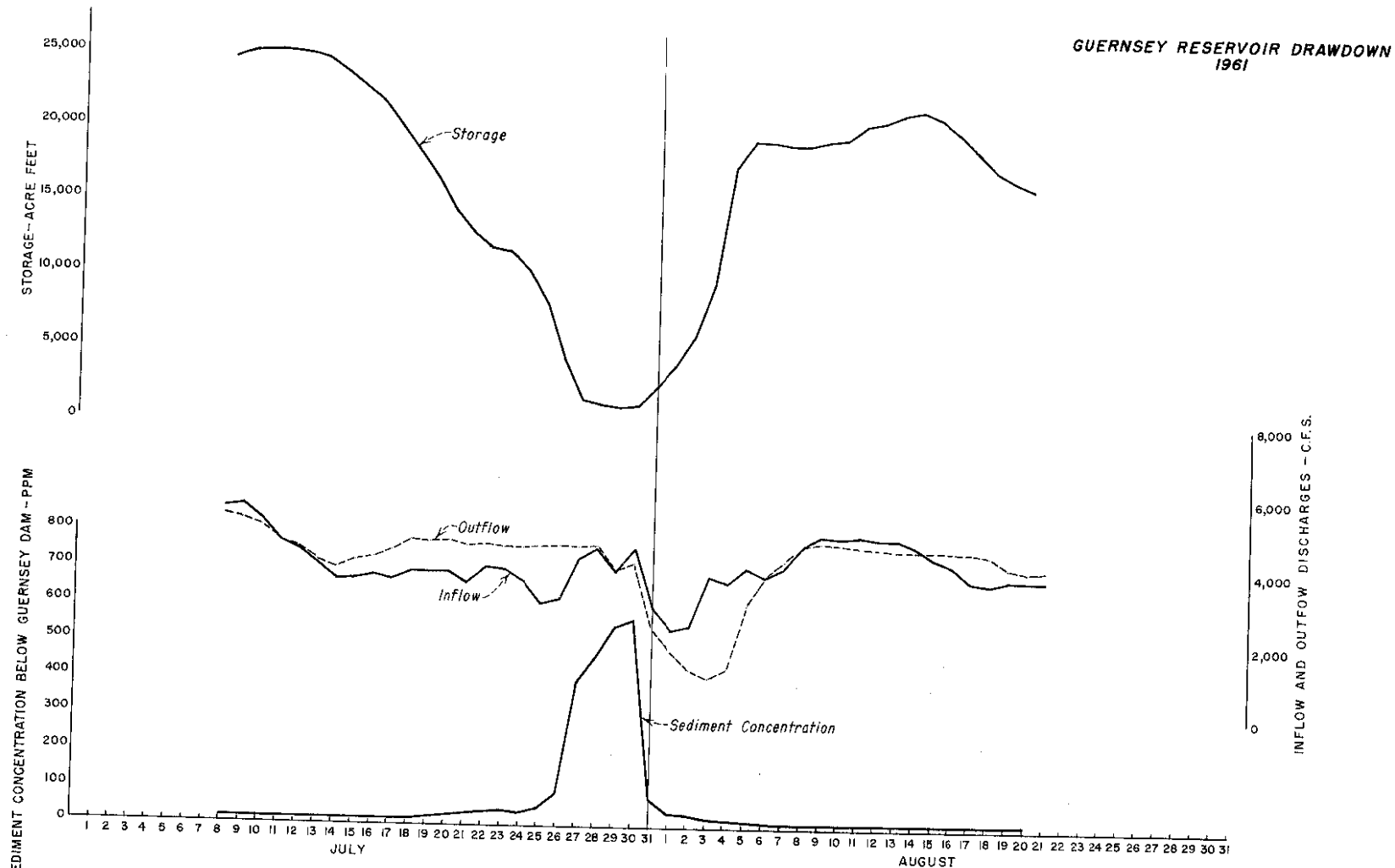


FIGURE 6. — Guernsey Reservoir drawdowns, 1961.

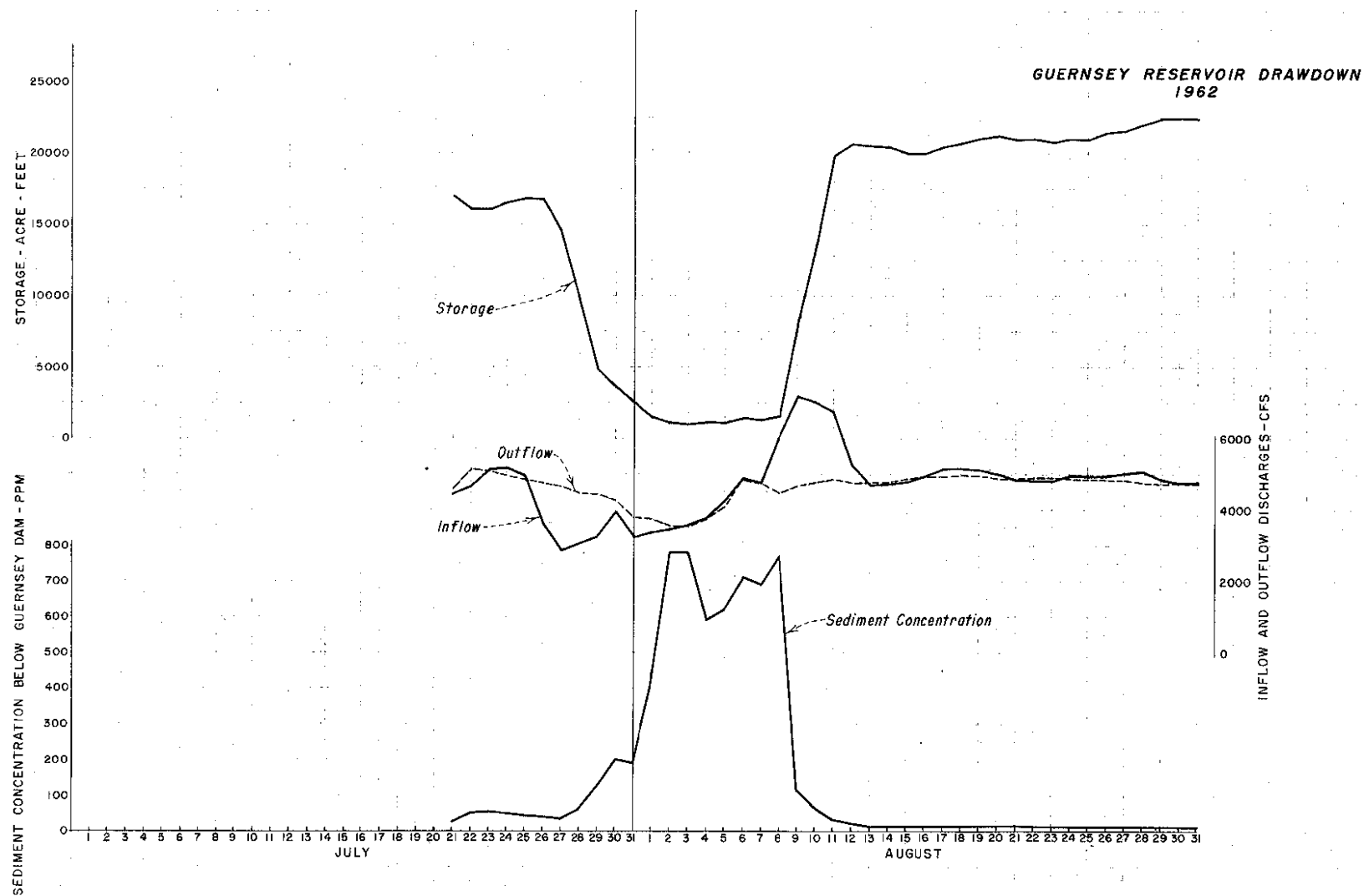


FIGURE 7.— Guernsey Reservoir drawdowns, 1962.

TABLE 3.—Drawdown rates for Guernsey Reservoir, 1960-62

Drawdown period ¹	Initial storage	Terminal storage	Change in storage	Draw-down rate	Maximum outflow concentration ²
	<i>Acres-feet</i>	<i>Acres-feet</i>	<i>Acres-feet</i>	<i>Acres-feet/day</i>	<i>Ppm per million</i>
July 10-12, 1960	16,900	5,500	11,400	5,700	84
Aug. 8-13, 1960	16,000	3,800	12,200	2,440	178
July 16-20, 1961	21,750	14,300	7,450	1,860	27
July 23-26, 1961	11,600	4,250	7,250	2,420	86
July 26-29, 1962	16,800	4,800	12,000	4,000	125

¹ The indicated periods were determined from the storage curves, figures 5, 6, and 7.

² The maximum outflow concentrations referred to in this table are the respective sediment concentrations for the terminal days of the defined drawdown periods.

relatively wide range of values for a given storage. It is also evident that maximum concentrations can be achieved with the reservoir storage below 2,000 acre-feet.

Effect of Inflow Fluctuation at Minimum Storage

A relationship to maximum outflow concentrations appears to occur between inflow fluctu-

ation at minimum storage, as shown in figures 5, 6, and 7. At minimum storage, an increase in inflow relative to outflow results in an increase in outflow sediment concentrations. Also, when the increasing relative inflow is reversed, there is a corresponding drop in concentration. It should be emphasized that these fluctuations of the inflow are "relative to the outflow discharge" and have significant effect only at minimum reservoir storage. This phenomenon is attributed to (1) maximum exposure of sediment beds resulting in maximum inflow concentrations, and (2) a minimum quantity of water (storage) acting as a diluent on the inflow concentrations, thereby providing higher concentrations available for withdrawal.

Future Sediment Withdrawals

The source of sediment that can be scoured from Guernsey Reservoir is sediment deposited above the dam since closure of the dam in 1928. With sediment inflow now curtailed by the upstream Glendo Dam, the sediment withdrawal

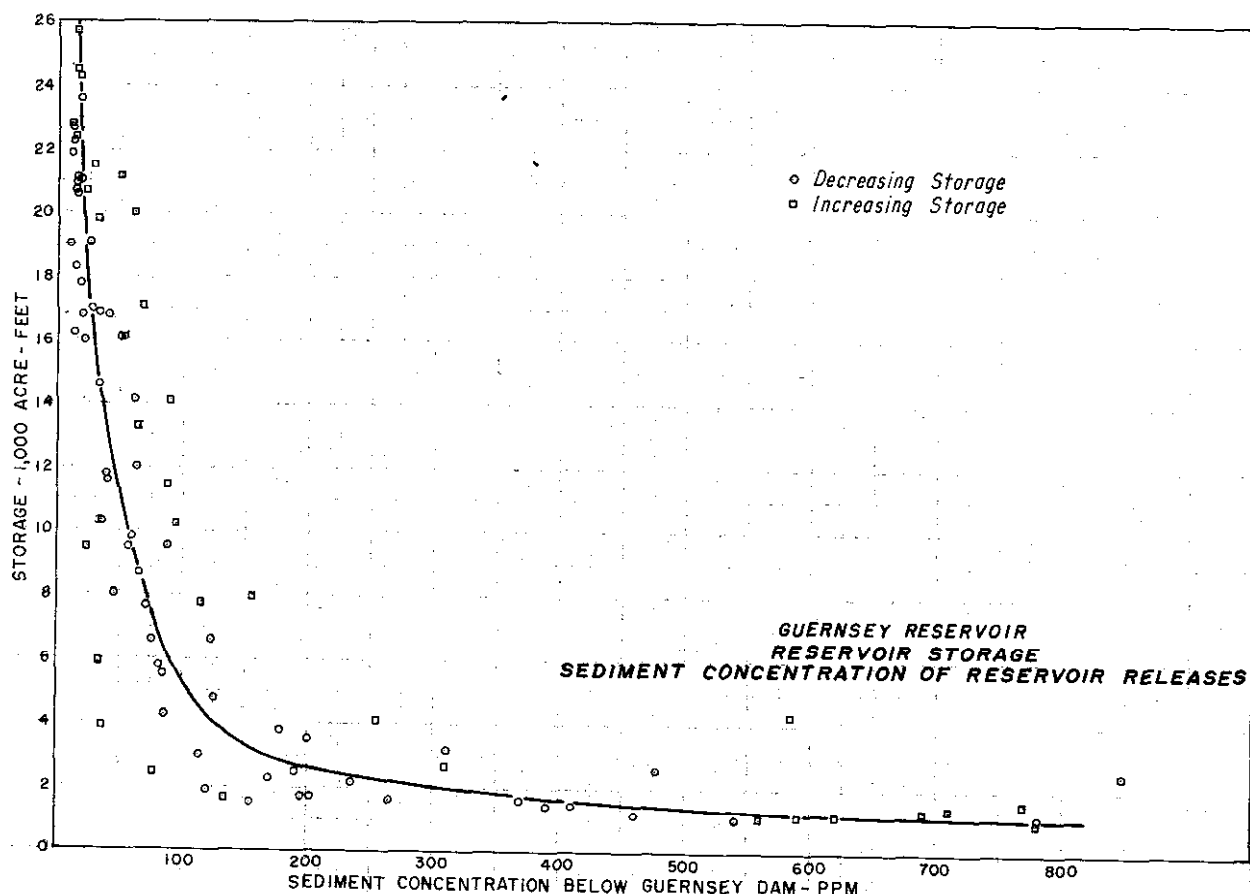


FIGURE 8. — Reservoir storage-sediment concentration of reservoir releases.

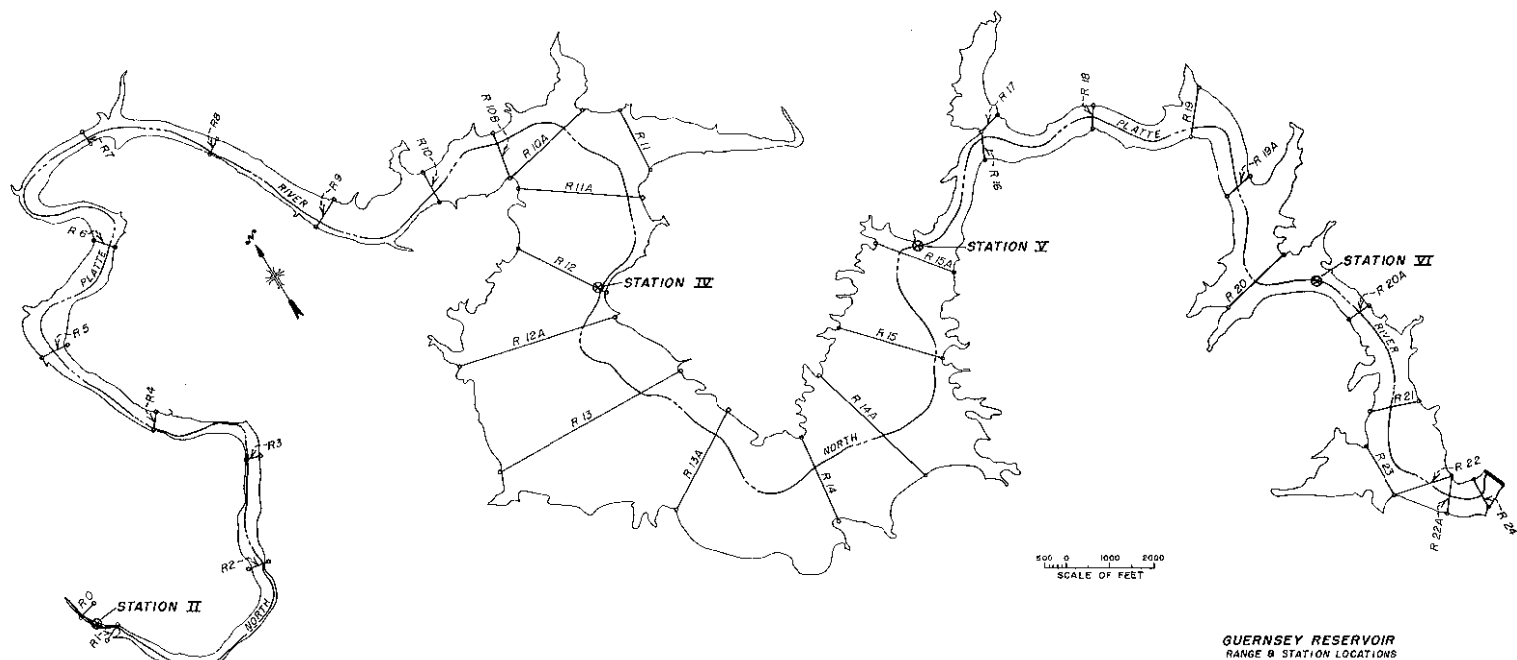


FIGURE 9.— Guernsey Reservoir range and station locations, May to July 1957.

investigations have shown that sediments can be flushed from the reservoir area. In analyzing the potential of future withdrawals from the reservoir, an appraisal was made to determine if the withdrawals of 1960, 1961, and 1962 show any decreasing rates of scour and to see if an estimate could be made of the physical limitations of future withdrawals.

Rate of Scour

Any analysis of the changing rates of sediment withdrawal from the reservoir during the 3 years of drawdown must include some appraisal of the sediment redistribution by areas within the reservoir. As shown on the reservoir topography (figs. 9, 10, and 11), there is considerable variation in location of sediment deposits within the reservoir. In the lower and upper parts of the reservoir, the deposits are confined between narrow canyons, while there is a wide flat area of deposition in the vicinity of sampling station IV and downstream to sampling station V. The suspended sediment samples collected at the sampling stations were used to compute the changes that took place during the drawdown periods. These changes in acre-feet between sampling stations are shown in table 4. Acre-feet values were determined by use of a unit weight of 67 pounds per cubic foot, which was supported by density samples of sediment deposits and a computed unit weight for deposited material of 22 percent clay, 62 percent silt, and 16 percent sand. Although it is realized the unmeasured load at each of the sampling stations would affect the results given in table 4, only those stations within the reservoir would have significant unmeasured load and the values shown should be adequate for comparative purposes.

To illustrate the results of the redistribution study, the changes shown in table 4 are plotted by reaches on figure 12. Since the volume of water varied for each drawdown period with 85,100 acre-feet for the July 10-19, 1960, period, 89,400 acre-feet for the August 8-18, 1960, period, 107,200 acre-feet for the July 20-August 3, 1961, period, and 179,000 acre-feet for the

TABLE 4.—*Sediment redistribution resulting from reservoir drawdowns*

Reach defined by sampling stations (1)	Deposition (+) or scour (—)			
	1960		1961	1962
	July 10-19 (2)	August 8-18 (3)	July 20- August 3 (4)	July 24- August 12 (5)
	<i>Acre-feet</i>	<i>Acre-feet</i>	<i>Acre-feet</i>	<i>Acre-feet</i>
II to IV...	—34.4	—40.5	—29.0	—10.3
IV to V...	—166.2	—45.6	—45.5	—101.6
V to VI...	+39.3	+9.2	—21.0	—40.4
VI to Dam.	+146.5	+66.2	+78.0	+107.7
Total...	—14.8	—10.9	—17.5	—44.6

July 24-August 12, 1962, period, the changes were converted to acre-feet per acre-foot inflow for comparative purposes. In general, the relationship, as given on figure 12, shows a decreasing rate of channel scour in the upper parts of the reservoir and a decreasing rate of deposition in the lower part of the reservoir.

Physical Limitations

The second factor considered in estimating the future sediment withdrawals from Guernsey Reservoir is the physical limitation on scour. Three possible interrelated conditions that appear to be acting as controls on scour are (1) the reservoir storage near the dam, (2) a limiting or controlling channel slope, and (3) a limiting channel width. The longitudinal profiles on figure 3 give some indication of the effect that reservoir storage and limiting channel slope would have upon future reservoir channel scour. The only data available on ultimate channel widths through the scouring portion of the reservoir were channel widths at sampling stations observed during the drawdowns of 1960, 1961, and 1962.

The controlling slope for ultimate channel degradation in the reservoir is supported by information shown on the longitudinal profiles (fig. 3). The slope of delta deposits in the reservoir of about 2.5 feet per mile, or 0.00047 foot per foot, for the 1957 thalweg was also observed at the time of the 1937 and 1947 surveys of Guernsey Reservoir. This was also

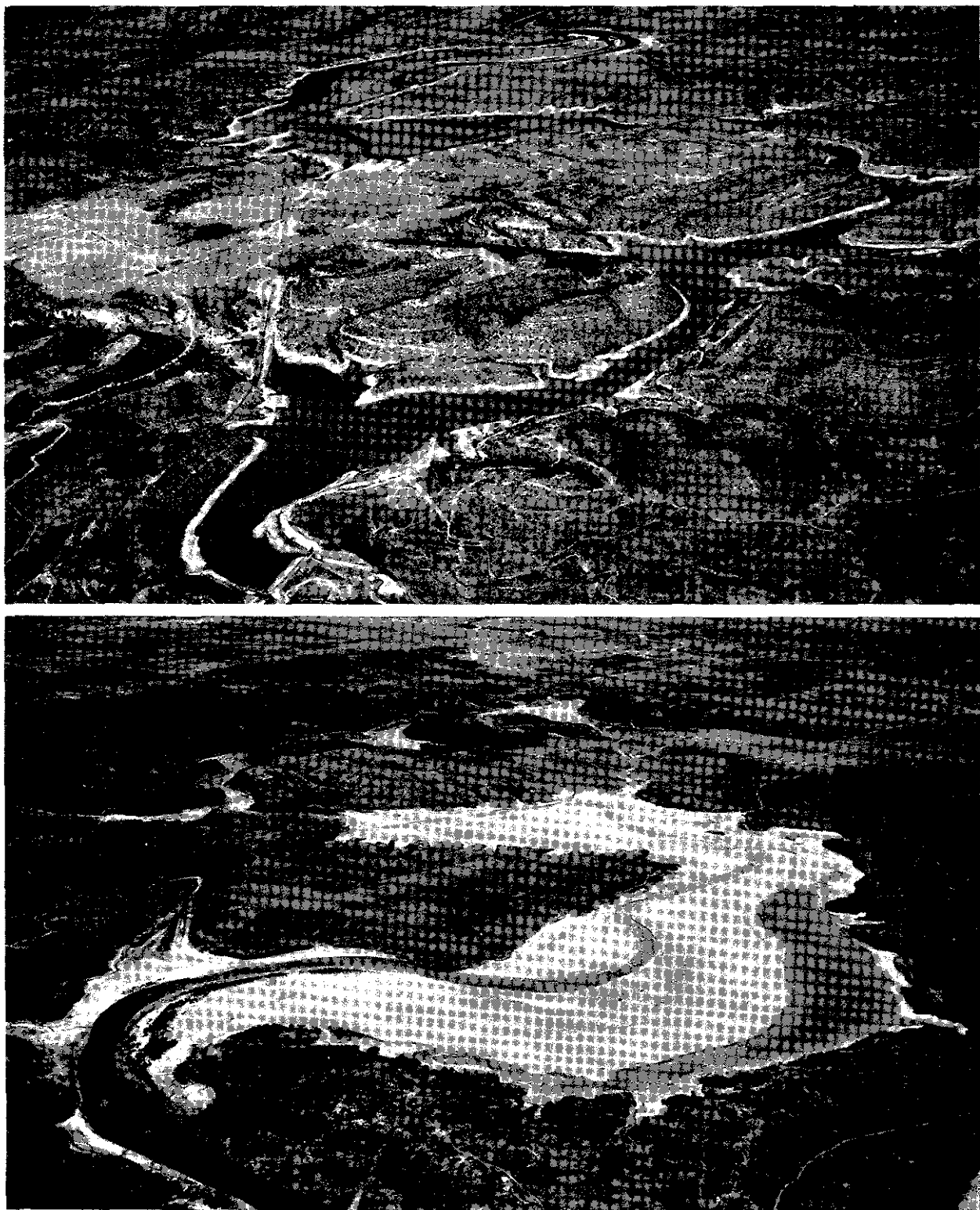


FIGURE 10.—Aerial views of Guernsey Dam and Reservoir: (top) during reservoir drawdown of July 14, 1960; and (bottom) looking southeast toward dam during drawdown. Sediment deposits between sampling stations IV and V are in foreground.

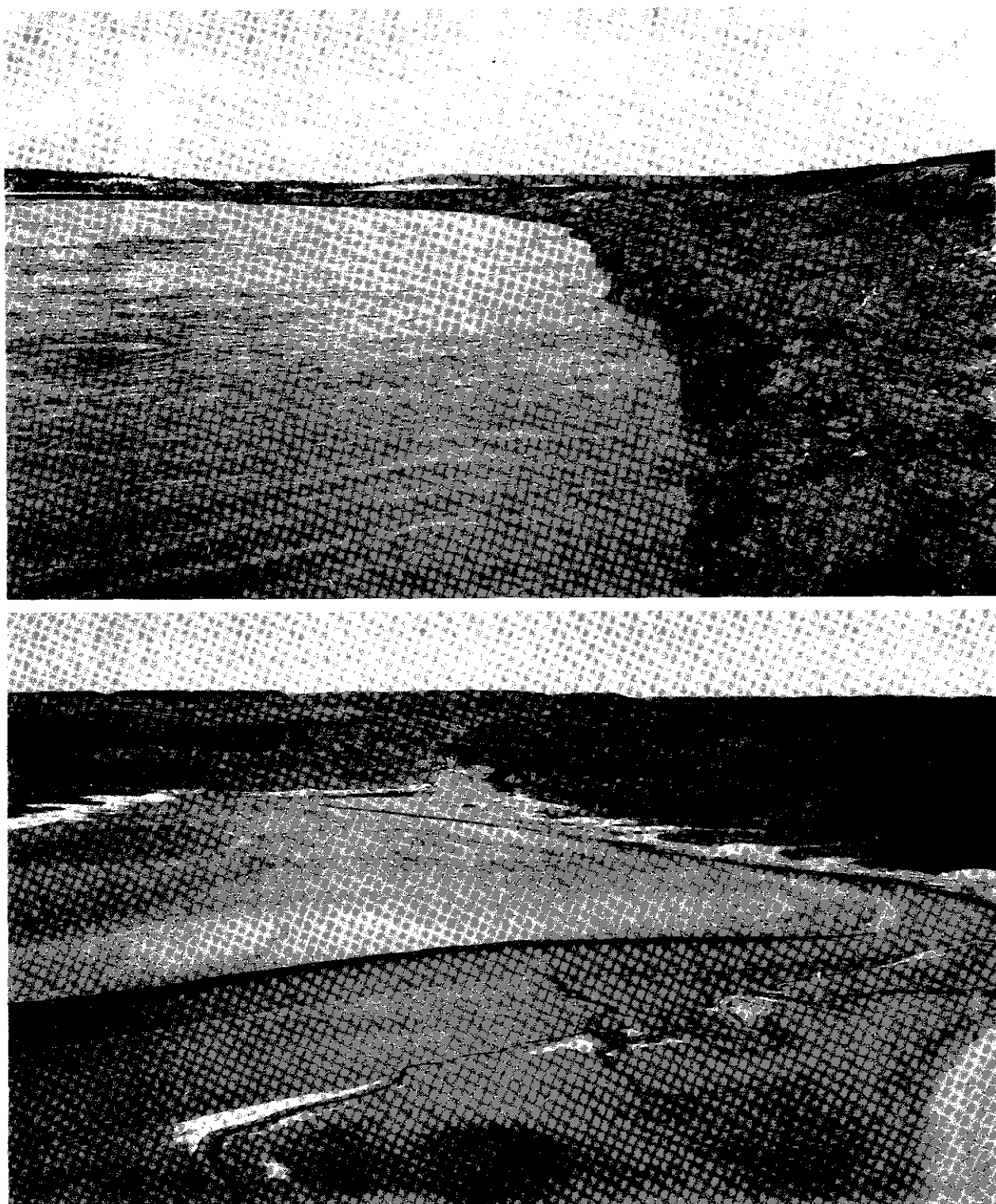


FIGURE 11. — (Top) North Platte River flowing through sediment deposits in Guernsey Reservoir during drawdown, August 1, 1962. Sloughing unstable banks are shown in reach between sampling stations IV and V. (Bottom) Aerial view of sediment deposits in area above sampling station V during drawdown July 28, 1961. This view shows channel scour and meander within the reservoir.

about the same slope as determined for the bed profile between sampling stations V and VI in 1962. An analysis of sediment samples collected in the delta area in 1957 and of bed material samples at sampling stations IV and V both give a mean bed material size of about 0.6 millimeter. A limiting slope of about 0.00047 also coincides with low transport rates for the reach above sampling station IV and a mean bed size of 0.6 millimeter. A control on the degradation within the reservoir will be the minimum reservoir level for future drawdowns. It is estimated that any future withdrawals will be patterned after those made in 1960-62. In this case, the controlling bed elevation in the vicinity of sampling station VI will be at about elevation 4375. Based upon the above observations, a predicted profile for ultimate reservoir channel scour is shown as figure 3.

The remaining factor of defining the controlling width for future reservoir channel scour was analyzed with respect to any observed changes in widths and stable channel widths. The observed channel widths at the sampling stations in the reaches of channel above the reservoir have varied from about 190 to 250 feet, with no obvious changes with time. However, there is evidence of more bank scour in the reach between sampling stations IV and V. An ultimate channel width, based upon a limiting tractive force of 0.1 pound per square foot, a D_{75} of 0.05 millimeter, and the ultimate slope of 0.00047, would be more nearly 600 feet.

The potential estimated material available for scour in Guernsey Reservoir is 4,700 acre-feet. This represents vertical scour down to the predicted controlling slope shown on figure 3 and lateral scour in the reach in the vicinity of sampling station IV and down to sampling station V. Of the above amount, an estimated 2,000 acre-feet is in the reach between sampling station IV and V and the remaining 2,700 acre-feet are above sampling station IV.

Predicting Future Sediment Withdrawals

Although the potential volume of sediment that can be withdrawn from the reservoir is

4,700 acre-feet, the 2,700 acre-feet of material above sampling station IV is predominantly material in the sand size function. Samples collected at sampling station IV show there had been a reduction in concentration with time to about 100 p.p.m. in 1962. It is concluded from this limitation in concentration at sampling station IV and the sand material in the delta deposits above station IV, that in computing sediment withdrawal from the reservoir the 2,000 acre-feet of material between sampling stations IV and V be considered as that available for scour from the reservoir. Based upon the average size analyses of deposited material, there would be about 1,700 acre-feet of sediment in this reach of silt and clay sizes.

In predicting the future scouring capabilities in Guernsey Reservoir, a rate of scour must be correlated with the 1,700 acre-feet considered as potentially available. If drawdowns in the future can be patterned after the drawdowns of the 3 years, 1960-62, the average rates of scour as shown on figure 12 should give some indication of the future rate. A projection of the data on figure 12, combined with the estimated physical limitation of scour as shown on figure 3, would indicate an average rate of about 0.00018 acre-foot per acre-foot of water inflow for future withdrawals. With withdrawals averaging 100,000 acre-feet of inflow per period, the average sediment scoured from the reservoir would be 18 acre-feet per drawdown period.

Conclusions

There is an estimated 1,700 acre-feet of material available in Guernsey Reservoir that could be scoured during properly regulated withdrawals.

Analysis of the withdrawals made during 1960, 1961, and 1962 shows that for effective scouring of reservoir sediment deposits, the reservoir should be drawn down to 2,000 acre-feet, or below. The relationship of sediment concentration below Guernsey Dam, with storage capacity shown on figure 8, defines the

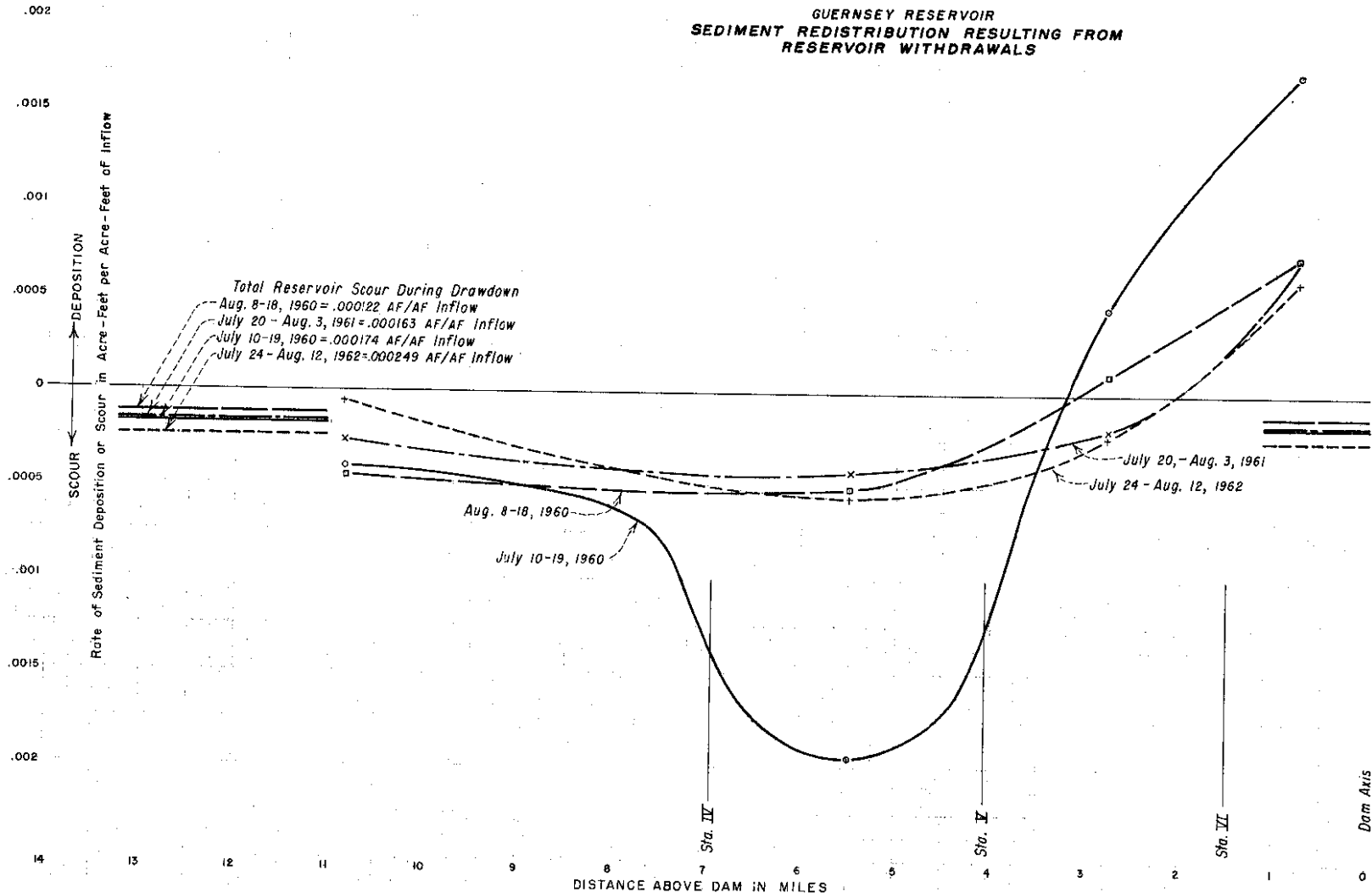


FIGURE 12.—Sediment redistribution resulting from reservoir withdrawals.

recommended storage that will result in given concentrations below the dam. This can be used to determine the best operation for obtaining the proper sediment concentration in the river relative to the requirement for the downstream distribution system.

Fluctuation of the reservoir inflow during a drawdown period appears to have an effect on sediment concentrations below Guernsey Dam. After a minimum reservoir storage has been reached, a fluctuation of about 1,000 to 2,000 c.f.s. at 1- to 2-day intervals should help maintain sediment concentration in the outflow from Guernsey Reservoir.

With drawdowns to 2,000 acre-feet capacity, or below, and by proper fluctuation of the reservoir, an estimated future sediment withdrawal rate of about 18 acre-feet appears possible for a 10- to 15-day period with an inflow of about 100,000 acre-feet. If it is assumed that sediment would be withdrawn from the reservoir twice annually for periods of 10 to 15 days each, then the estimated future annual sediment withdrawal rate would be approximately 36 acre-feet. This value, combined with an average annual withdrawal during the non-drawdown period, averaging about 6 acre-feet

per year as derived for the 1957-62 period, results in an estimated total annual outflow of about 42 acre-feet. The 1,700 acre-feet of material expected to be scoured within the reservoir and the small contribution of sediment from above Guernsey Reservoir should continue to give the desired sediment outflow for about 40 years. It is anticipated that the withdrawal rate will diminish with time; however, the amount of data accumulated is not sufficient to prognosticate as to the effect of diminishing withdrawal rate.

There can be expected to be an increase in the sand size material flushed from the reservoir, which has not been considered in this analysis. An additional detrimental effect will be the increased amounts of sand, as well as finer material in water flowing through the turbines.

Acknowledgments

The authors wish to express their appreciation to W. K. Lundgreen, soil and moisture conservation engineer, Torrington, Wyo., and E. L. Pemberton, hydraulic engineer, Office of Chief Engineer, Denver, Colo., both of the Bureau of Reclamation, for their cooperation and contributions to this paper.

DISCUSSION — SYMPOSIUM 4

Sedimentation in Reservoirs

Moderator: B. L. Hobbs, CE

Panel: L. C. Gottschalk, SCS
C. R. Miller, ARS
D. C. Bondurant, CE

MR. HOBBS:

Satisfactory engineering solutions to reservoir sediment problems require reasonably accurate forecast determinations of (1) quantities of material expected to be delivered to the reservoirs; (2) amounts of material expected to be carried through the reservoirs and discharged with water releases; (3) volumes of material retained; (4) average rates of sediment accumulations in channels tributary to reservoirs; and (5) depletion of reservoir storage space allocated for various purposes in designated elevation zones. Scheduled measurements are necessary to obtain basic information as required for operational purposes and for use in testing and improving methods of forecasting sediment quantities and distribution.

With these points in mind and in view of time limitations, the panelists are requested to confine the prearranged parts of your discussions to broad considerations involved in proj-

ect planning and design investigations and to provisions for periodic measurement of sediment accumulations.

MR. GOTTSCHALK:

On the subject of trap efficiency, I wish to direct attention to a definition of the term as "ability of a reservoir to retain sediment," where amounts of materials retained are expressed as percentages of the total quantities delivered to the reservoirs. The importance attached to trap efficiency in connection with project planning and design investigations has increased with the development of an entirely new retarding-type reservoir and with the increased knowledge of the interrelations of the numerous factors that affect sediment deposition and retention.

The first trap efficiency investigations were undertaken in the early 1930's by Carl B. Brown, who developed useful relations between trap efficiencies and the ratios of total reservoir capacities to drainage area sizes. However, it was found that these relations were not valid for all conditions, particularly those found in reservoirs located in the arid and semiarid areas in the United States. This led

to the investigations by Gunnar Brune, who developed a curve relating trap efficiency to the capacity-inflow ratios.

In addition, Brune attached considerable importance to (1) the "detention time" of sediment-laden water as a factor influencing deposition and (2) the related time factor involved in the decline of trap efficiency that starts at some stage during the period of storage depletion. Although investigators have recognized that numerous other factors influence trap efficiency, for the most part the early studies were based upon data obtained from single-purpose storage reservoirs where the time factors were found to be of primary importance.

New types of reservoirs have come into being as units in plans for accelerated development of water resources; these include the so-called dry reservoirs designed for operation to retard flood flows and reservoirs planned for operation to serve two or more purposes. In connection with investigations of these projects we have gradually accumulated evidence that, under certain conditions, some of the following factors affecting deposition may be equally as important as the interrelated time influences. The factors include reservoir shape; type of regulation; location of tributaries to the reservoir with respect to the outlets; sediment characteristics including mineral constituents as well as particle size; and chemical composition of the water as it affects the settling velocity of discrete particles.

In the case of the so-called "dry reservoirs," materials previously deposited tend to be "re-worked" and transported farther into or out of the reservoirs than is found in the permanent-storage reservoirs. Also, stilling effects during the initial periods of inflow, which tend to cause deposition before high inflow rates exceed the release rates, are generally not present in dry reservoirs. This is usually true to some degree at all stages of a flood when releases are equal to or exceed inflow rates. To illustrate the potential importance of this stilling effect upon the trap efficiency of a dry reservoir, we should take note of a comment by Robert F. Piest stated in paper No. 15 (p. 98) that on small watersheds, "... storms with a return period less than one year were the cause of 34 to 92 percent of the total sediment yield."

The Soil Conservation Service is constructing a type of "upstream retarding structure" at a rate of about 700 per year. The urgent need for information regarding trap efficiency

of reservoirs created by these structures has resulted in initiation of a plan for cooperative study by the Soil Conservation Service, Agricultural Research Service, and the U. S. Geological Survey, which provides for field measurements to obtain basic data. In conclusion, I suggest that further investigation be initiated to improve capabilities for estimating trap efficiency for all types of reservoirs.

MR. MILLER:

I am impressed with the coverage contained in the papers relating to sedimentation in reservoirs. This information represents an excellent documentation of the progress that has been made since the 1947 Conference, ranging from instrumentation and techniques for sediment measurement to methods developed to forecast conditions from 50 to 100 years in the future. In addition to the value of the papers as records of present knowledge and descriptions of engineering practices being used in connection with reservoir sediment problems, the papers will prove to be useful indicators of areas of need for further investigation and the proper direction of such efforts.

I consider the empirical methods for making forecast estimates of distribution of reservoir deposits, described by Heinemann,¹ and Borland and Miller,² to be generally satisfactory for predicting locations of the bulk of future deposits in connection with estimates of storage depletion. However, the methods presently available have many weaknesses and I consider them inadequate in several respects.

The use of reservoirs for recreational purposes is continuing at an accelerated pace, and demands for storage allocations to be used primarily for recreational activities are growing. We are becoming aware of the relations between the exposure of reservoir sediment accumulations and the number of mosquitoes. These developments have added to the importance associated with improving procedures for estimating sediment distributions corresponding to various regulation schedules. We expect probable errors in forecast estimates of distribution of the total deposits as determined by existing methods to be negligible insofar as storage depletions are concerned. However, differences in the location of relatively small fractions of the total may be quite important in connection with determinations relating to outlet designs and real estate takings. Also, drainage and ground water problems may be sensitive to small differences in sediment distribution. Papers presented by Mr. Ferral and Mr. Hadley³ and work by other investigators indicate progress is being made in studies of channel aggradation resulting from reservoir effects.

Our procedures for estimating sediment dis-

¹ See paper No. 83, p. 845.

² "Distribution of Sediment in Large Reservoirs," Amer. Soc. Civil Engin. Proc. 84 (No. HY2) Paper 1587, 1958.

³ See papers Nos. 44 (p. 376) and 80 (p. 806), respectively.

tributions could be improved; especially, we should refine procedures for predicting delta developments and progressive positions of accumulations near reservoir outlets.

MR. BONDURANT:

I believe the selection of papers has provided excellent coverage of reservoir sediment studies. These papers suggest the types and amounts of data that are required to serve various purposes; the types of surveys that should be conducted; the accuracies required in reservoir sediment measurements; and what measuring equipment and survey techniques can be used. I believe the information obtained by reconnaissance methods and spot measurements in many cases is entirely adequate for use as a qualitative indication of the extent of storage depletion, general magnitude of local problems, and as an aid to judgment concerning the need for and proper timing of more accurate instrumental surveys. The reconnaissance methods are particularly appropriate where sediment deposits are small and the primary interest concerns storage depletion. More detailed reservoir surveys are occasionally justified if the information is to serve as an aid in planning and designing other similar projects in the same region and as an aid to the optimum installation of permanent field facilities necessary for future sediment surveys in similar projects. The reliability of any survey, however, is largely dependent upon the judgment of the surveyor and, in the case of a reconnaissance type survey, it is particularly important that the assignment be given to an experienced man.

Insofar as distance measurements are concerned, there is a sufficiently wide range of good methods and techniques to allow a selection adaptable to any particular reservoir situation and type of survey one would want to make. The current meter method, first used for surveys of submerged areas by the Southwestern Division of the Corps of Engineers to determine distances of boat movement along a range line, gives good results in still water.

The use of a piano wire and reel mounted on a boat to measure distances from a shore position was first described in a TVA paper presented at the First Sedimentation Conference held in 1947. Since that time many improvements of the piano wire method provide more accurate data. These improvements include (1) the distance measuring wheel; (2) tensiometer devices to indicate the correct tension to be held in the wire; (3) floats to prevent sag; and (4) a distance-wheel attachment that automatically makes "fix" marks on a sonic depth recorder chart at intervals represented by 100

feet of boat travel, recently developed by the New Orleans District of the Corps of Engineers. This latter item is discussed in an article on pages 37 to 39 of the January-February 1962 issue of the *Military Engineer*. The "piano wire" method, with the wire permitted to sag to the bottom, has proved to be reasonably accurate for ranges of up to 15,000 feet. The error due to the sag is minimal on wide, shallow ranges and greater on narrow, deep sections, but a sag calculation will show that it is not generally as large as the errors inherent in other methods of distance measurement. The radist described by Isaac Shepherdson⁴ is accurate and perhaps the most satisfactory position-finding equipment that could be used under reservoir conditions involving extreme distances and depths.

I believe we should continue to improve the measuring equipment, because future needs cannot be foreseen. However, in the matter of normal range-type surveys of large reservoirs, good judgment in spacing of ranges for practicable surveys is one of the more important factors. In this connection, because of irregular reservoir boundaries and economic limitations placed upon the number of ranges that can be provided, merely increasing precision of profile surveys beyond that which can be attained by existing methods would not greatly improve the overall accuracy of reservoir capacity determinations and would be of questionable justification.

There is a dearth of dependable basic data regarding the densities of reservoir deposits and rates of compaction of materials of different characteristics deposited under the various conditions and subjected to varying conditions of submergence. Sediment load records of weight quantities and volumetric estimates of erosion are helpful in reservoir sediment investigations, but the degree of confidence to be placed in forecasts depends to considerable extent upon the soundness of the engineer's judgment in predicting future densities of the reservoir deposits. Accordingly, the improvement of our capabilities for making sediment-density observations is perhaps the most urgent present need insofar as field measurements of reservoir deposits are concerned.

I believe the development of the nuclear probe is the most important advancement for measurement of in-place densities of sediment deposits that has been made in many years. Densities indicated by the probe are considered to be quite accurate, but there are numerous operational obstacles that impose limitations on the practicability of making a desirable number of density measurements. These difficulties multiply with the increase in the size of the reser-

⁴ See paper No. 86, p. 869.

voir and water depths. Problems of penetrating deposits with densities exceeding 30 to 40 pounds per cubic foot are formidable, and the question regarding the extent to which the probing operation affects the density of the materials measured needs investigation.

MR. HOBBS:

We believe the foregoing statements provided adequate answers to some of the written questions, which were submitted prior to the panel session. The essence of questions directed to the panel members and authors of individual papers are presented below:

Mr. Miller, what approaches and procedures have been developed for predicting future delta formations with respect to shapes and extent of upstream deposition?

MR. MILLER:

As I stated earlier, tools for estimating future delta formations are inadequate. Some work has been done in an effort to develop step methods for predicting delta formations by applying sediment transport equations in combination with the available empirical methods used to obtain reasonable approximations of quantity distributions. This work, by employees of the Bureau of Reclamation, Corps of Engineers, and Agricultural Research Service, has not progressed to a point where publication of general predicting methods is warranted. Several papers dealing with various aspects of the delta formation problem have appeared in the literature since 1950. The paper, "Deposition at the Heads of Reservoirs," by A. S. Harrison that appeared in the Proceedings of the Fifth Hydraulics Conference, held at the State University of Iowa in June 1952, contains a comprehensive evaluation of the problem.

MR. HOBBS:

The gamma ray probe as presently developed from work originally sponsored by the Beach Erosion Board (Corps of Engineers) appears to be the best available means of determining the density of deposited reservoir sediment. The probe and accessories are expensive, however, and require special training and clearance for their use. What type of samples does the panel recommend for volume-weight determination in cases where the gamma probe is not available?

MR. BONDURANT:

A probe, of the same model as the one on display at the conference, is available at the Inter-Agency Sedimentation Project at St. Anthony Falls, Minneapolis, Minn., for loan to agencies

of the U.S. Government. At present it is advisable for anyone requesting a loan of the instrument to arrange for 1 or 2 days of instruction by someone familiar with its operation. It is anticipated that when an operation manual is finally prepared, there will be no particular need for special instruction. No license is required to use the probe, and with exercise of reasonable precautions it is not dangerous. However, it is advisable to check with State health agencies regarding their position on the use of the probe.

I have no suggestion regarding a type of sample to be taken in lieu of probe measurement. Perhaps the most comprehensive information regarding instruments for obtaining in-place soil samples is contained in a report entitled "Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes," by M. Juul Hvorslev, and published by the Waterways Experiment Station, Vicksburg, Miss., in 1948. However, the expense involved in sampling and analyzing sediment samples obtained by the instruments described, as well as the accuracy of the results, does not compare favorably with costs and results obtained by use of the nuclear probe. I suggest that arrangements for loan of the probe is simpler than many people realize.

MR. CALDWELL:⁵

The Corps of Engineers, Baltimore District, has developed a scheme to effect probe penetration to depths from 30 to 40 feet through deposits in the Baltimore Harbor. In this technique water is pumped through perforations in pipe used to suspend the probe and force it into the deposits — jets of water emerging from the perforations above the probe flow upward and reduce friction resistance between the sides of the pipe and the sediment.

DR. BROOKS:⁶

It seems that we have made a lot of progress in finding out about sediment in reservoirs, especially in surveys and so on. I would like to ask the panel, what are we going to do about sediment in the next century or two?

MR. GOTTSCHALK:

From the standpoint of design it is the practice of Federal agencies to provide sufficient reservoir capacity for sediment storage so that deposits will not encroach upon space allocated for useful purposes, during the period used for economic analysis of the project. In general, dredging is the only effective way to "desilt" a reservoir. In most cases in this country, however, it is not economical to restore capacity by desilting a reservoir; costs involved in dredging, hauling, and providing for disposal areas are usually prohibitive. There have been many

⁵ J. M. Caldwell, Beach Erosion Board.

⁶ Norman Brooks, California Institute of Technology.

suggestions for desilting reservoirs, but only a few have been effective in special cases and none have general application.

MR. MILLER:

In many cases, our economic system requires that reservoirs be designed with a view to eventual obsolescence.

DR. BROOKS:

We need a little more explicit answer and we should give more thought to the prospective problems of sediment in reservoirs three or four hundred years hence.

MR. SHARP:⁷

Although we are solving some problems concerning reservoir sediment now, we are losing sight of major long range problems that will eventually confront us. We should consider solutions to the problem of holding sediment at its source, where it represents productive soil; this solution to this problem is more important than the relatively short range problems associated with sediment after it reaches the streams. For example, degradation and aggradation are important on the Lower Colorado River, but the big problem involves such questions as: keep that thing from closing when 100,000,000 acre-feet of water per year when there is nothing but a mud flat above Hoover Dam?" and "In the Columbia River, how are we going to . . . how are we going to salvage 10,000,000 acre-feet of fresh water that goes down the river every year is diverted to the interior basins to irrigate land . . . ?"

MR. HOBBS:

The foregoing discussions represent abbreviated considerations of only a few of the problems relating to a single aspect of the sedimentation field. At the outset the problems were oriented primarily for benefit of the working engineer with an assignment. Accordingly, readers of the papers at this conference should consider this and not be misled by such limited treatment of the subject of reservoir sedimentation. For example, it should be understood that there are several reservoirs in different regions of the United States where storage depletion is expected to be negligible for long periods, ranging upward from 200 to 1,000 years, and possibly longer. In such cases it seems reasonable to expect that the structures creating some of these reservoirs probably will be obsolete before storage depletion becomes a problem.

It is obvious from the implications in the last question presented above and the statements which followed that authorities on the subject feel strongly that more thought should

be given to long range plans for (1) eventual use of partially filled reservoirs; (2) provision of substitute facilities; and (3) soil conservation measures that would reduce sediment yields. The panel members and I agree generally with these ideas, but we view planning as a continuing requirement and feel that any firm plan for reservoirs that extends far beyond periods used for current economic analyses would not be in accordance with present policies of the Federal Government.

If the thoughts expressed here serve to stimulate some further interest in the field, the discussions will have served their purpose.

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**NOTE: Page 933 is the last page of the
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