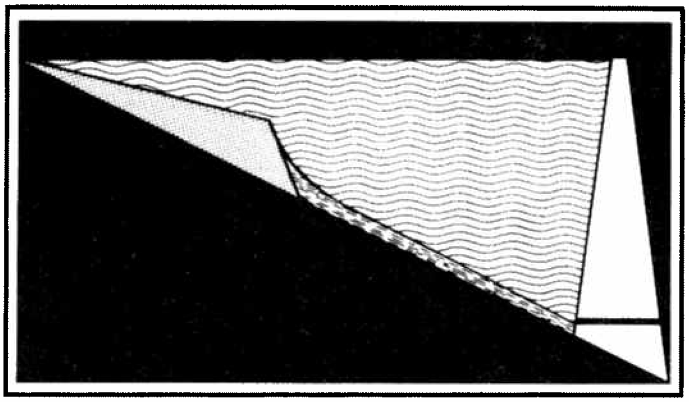


RESERVOIR SEDIMENTATION

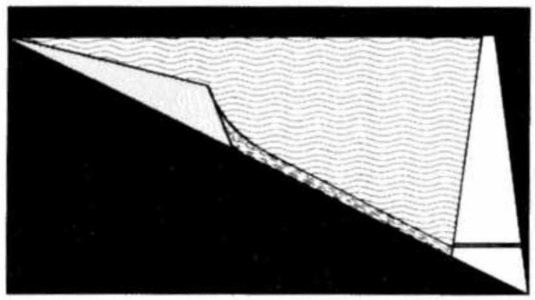
Technical Guideline for Bureau of Reclamation

U. S. Department of the Interior
Bureau of Reclamation

RESERVOIR
SEDIMENTATION

by

Robert I. Strand
Ernest L. Pemberton



TECHNICAL GUIDELINE FOR
BUREAU OF RECLAMATION

SI METRIC

SEDIMENTATION AND RIVER HYDRAULICS SECTION
HYDROLOGY BRANCH
DIVISION OF PLANNING TECHNICAL SERVICES
ENGINEERING AND RESEARCH CENTER

DENVER, COLORADO

OCTOBER 1982

CONTENTS

TABLES

FIGURES

RESERVOIR SEDIMENTATION

General. - All reservoirs formed by dams on natural water courses are subject to some degree of sediment inflow and deposition. The problem confronting the project planner is to estimate the rate of deposition and the period of time before the sediment will interfere with the useful function of the reservoir. Provisions should be made for sufficient sediment storage in the reservoir at the time of design so as not to impair the reservoir functions during the useful life of the project or during the period of economic analysis. The replacement cost of storage lost to sediment accumulation in American reservoirs amounts to millions of dollars annually (Chow, 1964).

There are a series of basic steps to follow in studying the sedimentation processes in reservoirs. First, sediment transported by the upstream river system into a reservoir is deposited and/or transported at a reduced rate further into the reservoir, the distance being dependent on the decreased water velocities. As sediment accumulates in the reservoir, storage capacity is reduced. The continued deposition develops distribution patterns within the reservoir which are greatly influenced by both operations of the reservoir and timing of large flood inflows. Deposition of the coarser sediments occurs in the upper or delta reaches while finer sediments may reach the dam and influence the design of the outlet works. A major secondary effect is the downsteam degradation of the river channel caused by the releases of clearer water.

These guidelines cover the essential sedimentation characteristics to be considered in the design of a dam and reservoir. The sediment related features requiring study are the sediment inflow, deposition, and degradation processes. Sedimentation processes in a reservoir are quite complex because of the wide variation in the many influencing factors. The most important being, (1) hydrological fluctuations in water and sediment inflow, (2) sediment particle size variation, (3) reservoir operation fluctuations, and (4) physical controls or size and shape of the reservoir. Other factors that for some reservoirs may be quite important are: vegetative growth in upper reaches, turbulence and/or density currents, erosion of deposited sediments and/or shoreline deposits, and operation for sluicing of sediment through the dam. The procedures described represent a combination of state-of-the-art together with methods that are practical, technically sound, and sufficiently varied to fit the complexity of the problem. It is because of this complexity that empirical relationships developed from surveys of existing reservoirs are being used to define sediment depositional patterns. Many mathematical models are being developed to simulate the physical processes of sediment transport and deposition in reservoirs. The models, to date, are not easily adapted to solve problems of reservoir sedimentation without some simplifications in defining the four most important factors previously described. With more research and additional reservoir survey data for verification of the mathematical models, they may become a useful method for predicting sediment deposition. Changes in these guidelines can also be expected in many of the empirical relationships with the continuing surveys of existing reservoirs. Further support to update these guidelines will occur as loss of storage capacity become more severe along with the economic and social changes affecting future reservoir uses.

In recent years, critical sediment problems have occurred in some of the reservoirs in all climatic regions of the world where complete loss of dependable storage resulted because of sediment deposition. In these situations, sediment control methods are being planned and, in many cases, construction completed on upstream sediment traps, bypass channels, special outlets for sluicing sediment, and mechanical dredging techniques. In many situations, sediment yields are high and conservation or erosion control measures in the drainage area are important for a reduction in the long-term sediment production. In the United States, these measures are usually carried out under the direction of the Soil Conservation Service, USDA, in cooperation with landowners and are encouraged by the Bureau of Reclamation.

Methods of determining sediment inflow. - Sediment is the end product of erosion or wearing away of the land surface by the action of water, wind, ice, and gravity. Water resource development projects are most affected by sediment which is transported by water. The total amount of onsite sheet and gully erosion in a watershed is known as the gross erosion. However, all the eroded material does not enter the stream system; some of the material is deposited at natural or manmade barriers within the watershed and some may be deposited within the channels and their flood plains. The portion of eroded material which does travel through the drainage network to a downstream measuring or control point is referred to as the sediment yield. The sediment yield per unit of drainage area is the sediment yield rate.

Most methods for predicting sediment yields are either directly or indirectly based on the results of measurements. Direct measurements of sediment yields are considered the most reliable method for determination of sediment yields. This is accomplished by either surveying of reservoirs or sampling the sediment load of a river, and both methods are described in subsequent sections of these guidelines. Other methods for predicting sediment yields depend on measurements to derive empirical relationships or utilize empirically checked procedures such as the sediment yield rate weighting factors or the Universal Soil-loss equation (Wischmeier and Smith, 1965).

a. Sediment yield rate factors. - The factors which determine the sediment yield of a watershed can be summarized as follows:

1. Rainfall amount and intensity
2. Soil type and geologic formation
3. Ground cover
4. Land use
5. Topography
6. Upland erosion (nature of drainage network-density, slope, shape, size, and alinement of channels)
7. Runoff
8. Sediment characteristics - grain size, mineralogy, etc.
9. Channel hydraulic characteristics

Some researchers have deemed it necessary to include some additional factors; however, even the nine above are interrelated. As an example, a heavy vegetative cover is dependent upon at least a moderate amount of rainfall; however, the ground cover conditions could be upset by tillage practices, overgrazing, or fire. Sediment transported from the drainage basin to a reservoir is controlled by the sediment transport characteristics of the river which is influenced by the first six factors but reflects a more direct combination of items 7, 8, and 9.

Systems of weighting the individual sediment influencing factors have been devised (Pacific Southwest Interagency Committee, 1968) to arrive at a sediment yield rate for an individual drainage basin. This type of analysis is best applied to preliminary planning studies and has its greatest reliability when the yield rates can be correlated with a measured sediment yield from an adjacent basin or subbasin.

An example of the techniques for weighting of the nine factors which is not identical but similar to those used in the report (Pacific Southwest Interagency Committee, 1968) is given in table 1. The weighted values would apply to the Pacific Southwest area, but because they are relative to each other, could be changed for other parts of the United States.

Table 1. - Rating chart of factors affecting sediment yield

Factors	Sediment yield level		
	High	Moderate	Low
1. Rainfall amount and intensity	10	5	0
2. Soil type and geological information	20	10	0
3. Ground cover	10	0	-10
4. Land use	10	0	-10
5. Topography	20	10	0
6. Upland erosion	25	10	0
7. Runoff	10	5	0
8. Sediment characteristics) 9. Channel hydraulics)	25	10	0

In computing the sediment yield of a drainage area above a dam or reservoir, a field inspection by a trained sedimentation specialist is needed to evaluate the factors in table 1 for weighting the significance of the nine factors affecting sediment yield. Upon completion of an inspection by the specialist, recommended procedures will be given on (1) available data and methods for analyzing data, (2) techniques available for predicting sediment yields in gaged as well as ungaged drainage basins, and (3) additional measurements required to compute sediment yields.

A well-known method for determination of sediment yields from small drainage areas is the empirical relationship developed by Wischmeieir and Smith (1965), most commonly referred to as the Universal Soil-loss equation. It should be recognized that gross erosion determined by this empirical method is, at best, an approximation and considered a rough estimate. It is normally applied to areas of less than about 4 mi^2 (10 km^2) and even then may have to be corrected by a sediment delivery ratio when converting gross erosion to sediment reaching a main river channel.

b. Reservoir resurvey data. - Measurement of the sediment accumulation in a reservoir is considered by many engineers as the best method for determining the sediment yield. Surveys of existing reservoirs for determining loss of storage space and distribution of sediment deposits within the reservoir provide data on sediment yield rates as well as for operations purposes. It is important that when construction is completed on a dam, a plan be established for surveying or monitoring of the sediment accumulation. Even before construction of the dam is completed, a decision is needed on the basic method selected for future surveys and technique for analyzing sediment accumulation (Blanton, 1982).

The main purpose of a reservoir survey is to determine the storage capacity at the time of the survey which when compared to an earlier survey (usually the original survey) gives the sediment accumulation. The storage volume computations are made from an area-capacity computer program involving computation of capacities corresponding to each elevation in the area-elevation data set and fitting the capacity-elevation relationship using either cubic spline or least square set of equations (Bureau of Reclamation ACAP Program). The end product of the area-capacity computations is the plot of the areas and capacities for the original and new surveys. An example of this plot is shown in figure 1.

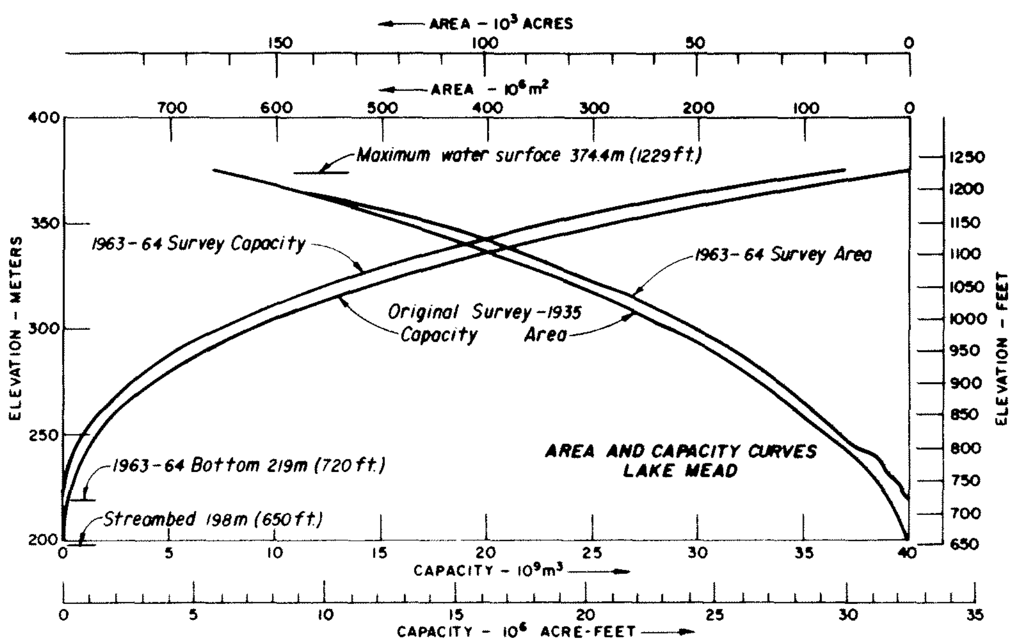


Figure 1. - Area and capacity curve for Lake Mead.

A comparison of capacities between the two surveys as shown in figure 1 gives the measured volume of sediment accumulation. It is important in this sediment volume computation that the method selected to compute capacities from contour areas be the same for both of the surveys being compared. That is, if the ACAP method is used for computing the resurveyed reservoir capacity, it should also be used for the original capacity computations. This would help eliminate any differences in technique having undue influence on the sediment volume computations. All information from the survey should be documented in the Reservoir Sedimentation Data Summary sheet which is provided to the Subcommittee on Sedimentation, Interagency Advisory Committee on Water Data for use in the periodically published summary on reservoir surveys (U.S. Department of Agriculture, 1978).

Other worthwhile analyses of data from reservoir sedimentation surveys are to make a plot of percent reservoir depth versus percent sediment deposit or to plot a sediment deposition profile throughout the length of the reservoir. The plot of percent depth versus percent sediment (fig. 2) has been used in developing design curves in predicting the distribution of sediment deposits in planning studies. The deposition profile provides valuable information for defining the delta, foreset slopes for possible density currents, and depth of sediment depositions at the dam. An example of a dimensionless plot of a sediment deposition profile for Lake Mead is shown in figure 3.

At the time of the reservoir survey, data are also needed on some of the characteristics of the sediments both as deposited and moving through the reservoir. Samples of deposited sediments should be spaced throughout the reservoir area to be representative of deposits in the topset and foreset slopes of the delta as well as at the bottomset slopes in the deeper parts of the reservoir. Analysis of the samples collected consists of density, particle-size distribution and mineralogic composition. These data on deposited sediments are used for a better understanding as to the source of incoming sediments, for use in study of density currents or study of sluicing capabilities through outlet works, for verification of models being developed on movement of sediment through reservoirs, and for development of empirical relationships to be used in the planning and design of other reservoirs. In addition to the above uses, data on sediment characteristics when combined with survey data on depths of sediment near the dam can be used to identify future problems of sediment deposition associated with inflow to powerplant intakes or plugging of outlet works. A unique sediment deposition problem to be evaluated in reporting the results of the survey data is the effects of bank sloughing, landslides, and valley wall erosion by wave action or unstable slopes.

Reservoir survey data (U.S. Department of Agriculture, 1978) provide an excellent source for determining sediment yield rates for any part of the United States. Adjustments in the sediment yield rate will usually be necessary to account for variation in drainage area characteristics. One of the most important variations is the size of the drainage basin. Some investigators have found that the sediment yield varies with the 0.8 power of the drainage area size (Chow, 1964) (equivalent to sediment yield rate varying with -0.2 power of the drainage area). Figure 4 is a plot of sediment yield rate versus drainage area which was developed from selected reservoir resurvey data in the semiarid climate of southwestern United States. In using the drainage area versus sediment yield relationship as shown on figure 4, it is best to make a calibration with a known sediment

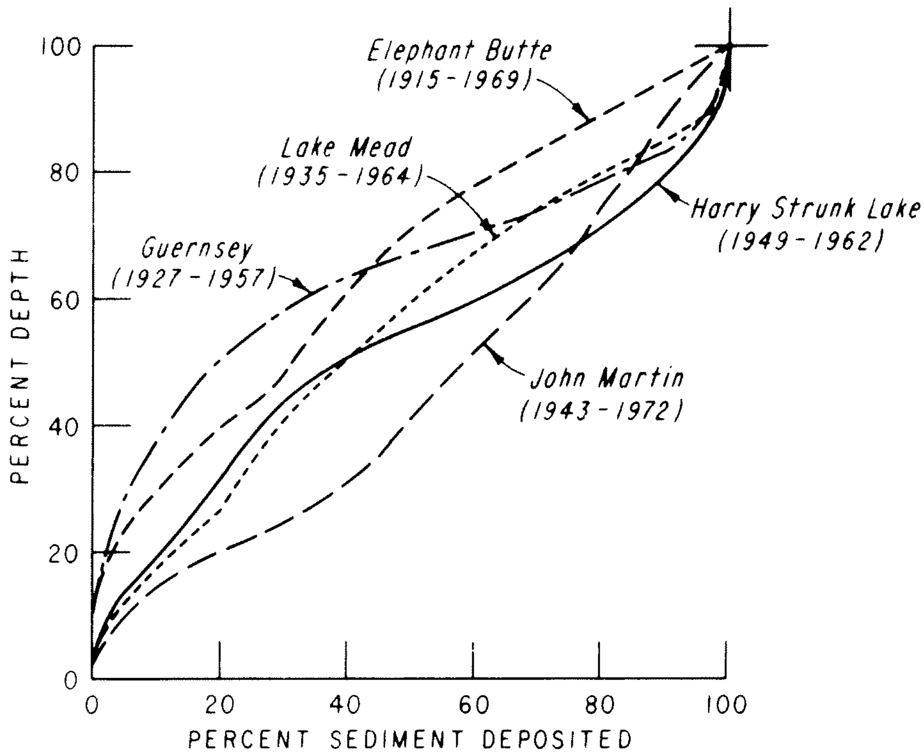


Figure 2. - Sediment distribution from reservoir surveys.

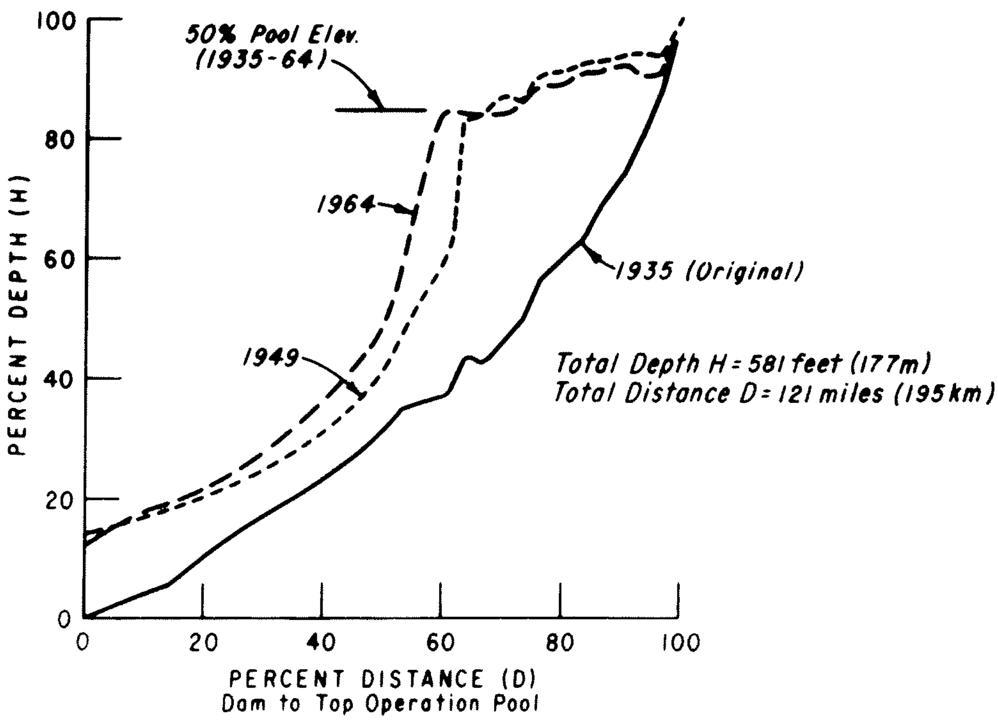


Figure 3. - Lake Mead sediment deposition profile.

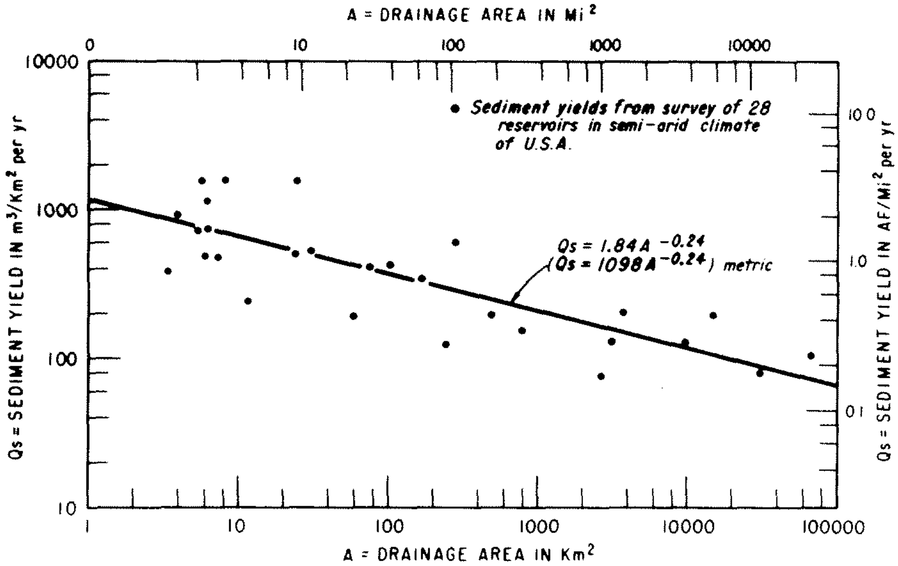


Figure 4. - Average annual sediment yield rate versus drainage area size.

yield and evaluate the nine sediment contribution factors. This calibration, along with an identification of similar sediment contributing characteristics, will permit drawing a parallel line similar to that shown on figure 4 through any measured data point.

c. Sediment sampling data. - Sampling is the surest method of obtaining an accurate determination of the suspended sediment load being carried by a stream at a particular location. Suspended sediment sampling in combination with total load computations is the preferred method used for planning studies in determining the sediment inflow to a proposed reservoir. The objective of a sediment sampling program on a river is to collect sufficient samples of sediment carried both as suspended load and as bedload to define the total sediment being transported. For suspended sediment sampling it is essential to measure the water discharge, Qw in ft^3/s (m^3/s) which is combined with suspended sediment concentration, C, in mg/L to give the suspended sediment load Qs in tons/day by the equation:

$$Qs = 0.0027\ C\ Qw \text{ (inch-pound units)} \tag{1}$$
$$\text{or } Qs = 0.0864\ C\ Qw \text{ (metric units)}$$

Suspended sediment sampling equipment and techniques for collecting can vary considerably depending on program objectives and field conditions. Suspended sediment sampling devices are designed to collect a representative sample of the water-sediment mixture. A thorough discussion of sediment samplers and techniques for sampling is given in either the series of reports prepared by U.S. Interagency Sedimentation Project (1940 to 1981) or in the U.S. Government Handbook (1978). An example of the U.S. Interagency Sedimentation Project designed sampler is shown in figure 5.

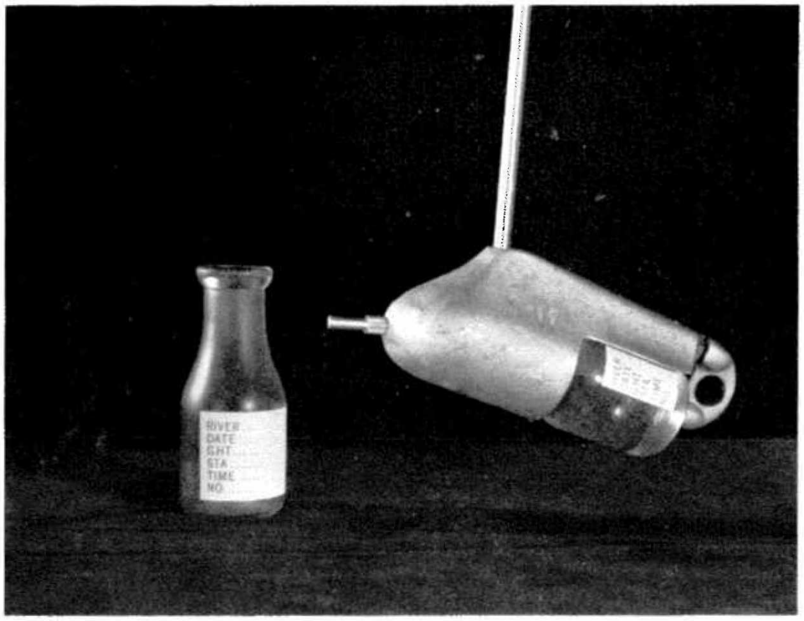


Figure 5. - Suspended sediment sampler DH-48.

In the collection of suspended sediment samples, it is important that samples represent an integration with width across the channel as well as with depth from the water surface to the streambed. Although other methods for sampling are described in the U.S. Government Handbook (1978), the EWI (equal-width-increment) method provides the most representative sample of the total suspended sediment load. It is accomplished by sampling at equally spaced widths or increments across the cross section and maintaining a constant travel rate in each of the verticals sampled. In this method, a composite sample is made of all verticals sampled for only one laboratory analysis of sediment concentration in mg/L and particle-size distribution.

The sediment sampling program will vary from one river to another, depending on temporal variations in the sediment load and particle-size distribution of the suspended and bed material sediments. The frequency of sampling suspended sediments will usually vary from daily samples to once or twice a month but should always include samples during the flood events. In many situations, the collection and analysis of suspended sediment samples is an expensive process, and daily sampling yields a good deal of duplication through a base flow period. For these reasons, the once or twice a month or miscellaneous sampling which includes sample of flood flows is more common and economical.

The objective of any suspended sediment sampling program is to develop a correlation between water discharge and sediment load commonly called a suspended sediment rating curve. This rating curve is normally a plot on logarithmic paper of water discharge Qw in ft^3/s (m^3/s), versus sediment load, Qs in tons/day from equation 1. These curves can best be computed by least squares analysis with water discharge as the independent variable usually defined by one to three such relationships. When two or three equations are computed from the plotted points, the extrapolation beyond the observed data, especially at high flows, is considered more reliable because the skewing effect of the data points at the other extreme has been eliminated. It is important in this extrapolation that maximum concentrations of sediment be considered to avoid the potential hazard of extrapolating beyond either an observed high value for the stream being sampled or no greater than about 50 to 60 percent concentration by weight.

The one to three equation procedure can also be adjusted so that a second, parallel set of curves will produce the sediment load equal to the sum of the observed data points. The result of this procedure for computing suspended sediment rating curves is shown in figure 6 where the equation for any segment is in the form:

$$Q_s = a\,Q_w^b \qquad (2)$$

in which Q_s = suspended transport tons/day
Q_w = discharge, ft^3/s (m^3/s)
a = coefficient
b = exponent

RIO TOA VACA NEAR VILLALBA, PUERTO RICO

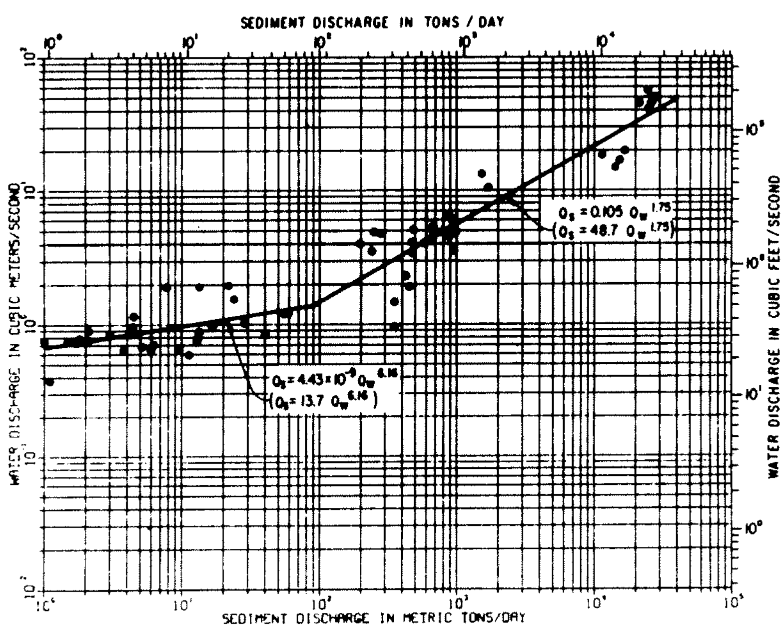


Figure 6. - Suspended sediment rating curve for Rio Toa Vaca near Villalba, Puerto Rico.

An approximate 5-year sampling period may be needed to adequately cover the full range in water discharges and to avoid extreme curve extrapolation. However, a shorter period may be possible if the range in flows is adequately covered. The upper portion of the rating curve is most critical; it significantly affects the rate of sediment transport because of the extreme large sediment loads carried during flood periods. Another variation in rating curves is described by Miller (1951) when the source of runoff can be a combination of either snowmelt or rainstorms. It may be necessary to develop individual sediment-rating curves for each of the seasons. Runoff from thunderstorms will usually transport sediment at higher concentrations than runoff from snowmelt taking place in the higher elevations.

Suspended sediment rating curves can be combined with available water discharge records to determine the long-term average sediment yield. The longer the period of discharge records, the more reliable the results. One technique for gaging station records that cover a long period is to construct a flow-duration curve from the daily water discharges. This curve is really a cumulative frequency plot that shows the percent of time that specific discharges are equaled or exceeded for the period of record. For some streams, where only short-term discharge records are available, a long-term flow-duration curve can be computed from a correlation of short-term to long-term records at a gaging station either on the same stream or nearby stream. If the flow-duration curve is representative of the long-term flow of the stream, it may be considered a probability curve and used to represent future conditions. With this assumption, it is combined with the suspended sediment-rating curve as described by Miller (1951) to determine the long-term average suspended sediment yield for any projected period such as 100 years. An example of the flow-duration curve for the same station used to develop the sediment rating curve on figure 6 is illustrated in figure 7. Reclamation's (Bureau of Reclamation) computer facility has linked up with that of the U.S. Geological Survey in Reston for obtaining flow-duration data for any desired period of flow record. Table 2 shows the computation of suspended sediment load at the gage based on combining the sediment rating curve with the flow-duration curve.

d. Unmeasured sediment load. - To analyze the unmeasured portion of the total sediment load requires a knowledge of the following terms:

Bed material. - The sediment mixture of which the streambed is composed.

Bedload. - Sediment that moves by rolling or sliding on or near the streambed.

Bed material load. - That part of the sediment load which consists of grain sizes represented in the bed.

Wash load. - That part of the sediment load which consists of grain sizes finer than those of the bed.

Suspended load. - Particles moving outside the bed layer.

Unsampled zone. - The 3 or 4 inches (7.62 to 10.2 cm) from the streambed up to the lowest point of the sampling vertical. Most suspended sediment samplers cannot sample within this zone.

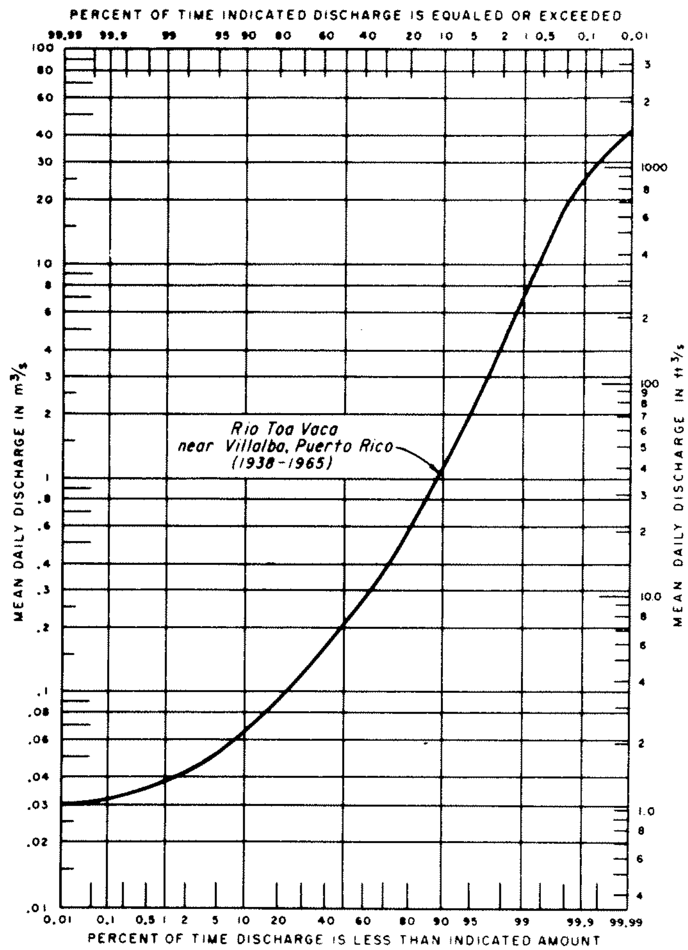


Figure 7. - Flow duration curve for Rio Tao Vaca near Villalba, Puerto Rico.

The suspended sediment load as computed in table 2 represents only a portion of the total sediment load. The unmeasured load consists of bedload plus suspended sediments in the unsampled zone between the sampler nozzle and the streambed. At the time the sediment sampling program is established, a preliminary appraisal is made on the percentage that the unmeasured load is of the total load. A useful guide for evaluating the unmeasured load is the bedload correction shown in table 3. Five conditions are given for defining bedload dependent upon suspended sediment concentration and size analysis of streambed and suspended materials. As shown in table 3, either condition 1 or 2 may result in significant bedload which would require a special sampling program for computing the unmeasured sediment load. Conditions 3, 4, and 5 usually indicate a 2 to 15 percent correction factor which would not require any special bedload sampling program.

Table 2. - Sediment load computations of Rio Toa Vaca near Villalba, Puerto Rico

Project Puerto Rico Reservoir Toa Vaca
Stream Rio Toa vaca Section ______ Date May 1968
Period of record: Streamflow 1938-1965 Sediment 1963-1967
Computed by ______ Checked by ______

1	2	3	4		5		6		7	
LIMITS %	INTERVAL	MIDDLE ORDINATE	Q_w		Q_s		$Q_w = \frac{\text{Col. 2} \times \text{Col. 4}}{100}$		$Q_s = \frac{\text{Col. 2} \times \text{Col. 5}}{100}$	
			ft^3/s	(m^3/s)	ton/d	(ton/d)				
0.00 - 0.02	0.02	0.01	1,412.0	(40.0)	34,151	(30,984)	0.282	(0.008)	6.83	(6.20)
0.02 - 0.1	0.08	0.06	1,037.8	(29.4)	19,925	(18,077)	0.830	(0.024)	15.94	(14.46)
0.1 - 0.5	0.4	0.3	617.8	(17.5)	8,038	(7,292)	2.47	(0.070)	32.15	(29.17)
0.5 - 1.5	1.0	1.0	250.6	(7.1)	1,657	(1,504)	2.51	(0.071)	16.57	(15.04)
1.5 - 5.0	3.5	3.25	115.4	(3.27)	426.6	(387)	4.039	(0.114)	14.93	(13.55)
5 - 15	10	10	44.1	(1.25)	59.7	(54.2)	4.410	(0.125)	5.97	(5.42)
15 - 25	10	20	20.8	(0.59)	0.583	(0.531)	2.080	(0.059)	0.058	(0.053)
25 - 35	10	30	14.1	(0.40)	0.053	(0.049)	1.410	(0.040)	0.005	(0.005)
35 - 45	10	40	10.6	(0.30)	0.009	(0.008)	1.060	(0.030)	0.001	(0.001)
45 - 55	10	50	8.1	(0.23)	0.002	(0.002)	0.810	(0.023)	0.0002	(0.0002)
55 - 65	10	60	6.4	(0.18)	0.0004	(0.0004)	0.640	(0.018)		
65 - 75	10	70	4.6	(0.13)			0.460	(0.013)		
75 - 85	10	80	3.5	(0.10)			0.350	(0.010)		
85 - 95	10	90	2.4	(0.068)			0.240	(0.007)		
95 - 96.5	3.5	96.75	1.7	(0.048)			0.060	(0.002)		
98.5 - 99.5	1.0	99.0	1.3	(0.038)			0.013	(0.0004)		
99.5 - 99.9	0.4	99.7	1.2	(0.034)			0.005	(0.0001)		
99.9 - 99.98	0.08	99.94	1.1	(0.030)			0.001			
99.98 - 100	0.02	99.99	1.1	(0.030)			0.0002			
						Total	21.67	(0.615)	92.45	(83.90)

Annual discharge = Total Q_w 21.67 x 365 x 1.9835 = 15,700 acre-ft/yr
= (Total Q_w 0.615 x 365 x 86.4 x 10^3 = 19.4 10^6 m^3/yr)

Annual sediment load = Total Q_s 92.45 x 365 = 33,700 ton/yr
= (Total Q_s 83.90 x 365 = 30,600 ton/yr) (metric)

$$\text{Average concentration, } C = \frac{Q_s}{Q_w \times 0.0027} = \frac{92.45}{21.67 \times 0.0027} = 1580 \text{ mg/l}$$

$$= \left(\frac{Q_s}{Q_w \times 0.0864} = \frac{83.90}{0.615 \times 0.0864} = 1580 \text{ mg/l} \right)$$

A special sampling program to be undertaken under conditions 1 and 2 in table 3 is usually established for total sediment transport computations by use of the Modified Einstein procedure (Colby and Hembree, 1955; Bureau of Reclamation, 1955 and 1966). Modified Einstein computations require the collection of the following data for at least 5 to 10 discharges covering the range of flows with as many measurements at higher discharges as possible:

Discharge measurements: Cross-section area, channel width, depth, mean channel velocity, and streamflow

Sediment samples: Suspended sediment samples analyzed for concentration and size distribution, bed material samples analyzed for size distribution, and water temperature

Table 3. - Bedload correction

Condition	Suspended sediment concentration mg/L	Streambed material	Texture of suspended material	Percent bedload in terms of suspended load
1/ 1	<1000	Sand	20 to 50 percent sand	25 to 150
1/ 2	1000 to 7500	Sand	20 to 50 percent sand	10 to 35
3	>7500	Sand	20 to 50 percent sand	5
2/ 4	Any concentration	Compacted clay gravel, cobbles, or boulders	Small amount up to 25 percent sand	5 to 15
5	Any concentraton	Clay and silt	No sand	<2

1/ Special sampling program for Modified Einstein computations required under these conditions.
2/ A bedload sampler such as the Helley-Smith bedload sampler may be used or computations made by use of two or more of the bedload equations when bed material is gravel or cobble size.

Table 4. - Modified Einstein procedure computation

O U T P U T

DETERMINATION OF TOTAL SEDIMENT LOAD IN A STREAM

JOBIDENT NIOBRARA RIVER- RIVER RANGE 5

METHOD OF COMPUTATION MODIFIED EINSTEIN DATE OF COMPUTATION 03/26/82

DATE OF SAMPLE 06/13/79 TIME OF SAMPLE TEMPERATURE 73.0 F (22.2 C) SLOPE OF ENERGY GRADIENT .00130 FT/FT (.00130 M/M)

DISCHARGE 850. CFS (24.1 M^3/S) CONCENTRATION IN PPM 296. SAMPLED SEDIMENT 679. TONS/DAY (616. TONS/DAY)

D65 = .3080 MILLIMETERS D35 = .2360 MILLIMETERS

AREA 538. FTSQ. (50. M^2) TOP WIDTH 705.0 FT. (215. M) EQUIV. DEPTH 0.00 FT. EQUIV. SLOPE = 0.00000 FT/FT

VELOCITY 1.58 FT/SEC. (.482 M/S) EQUIV. WIDTH 0.0 FT. AVERAGE BOTTOM DEPTH .76 FT. (.232 M) HYDR. RADIUS = .76 FT. (.232 M)

DISTANCE BETWEEN SAMPLER AND BED (DSUBN) .30 FT. (.0914 M) AVERAGE DEPTH FROM SAMPLE VERTICALS (DSUBS) .76 FT. (.232 M)

SIZE FRACTION IN MILLIMETERS		PERCENT OF MATERIAL SUSPENDED	BED	IBQB T/D	QPRIME SUBS(T/D)	Z - V A L U E S COMPUTED	FITTED	COMPUTATIONAL FACTORS F(J)	F(I)+1	COMPUTED TOTAL LOAD T/D	T/D
.0160	.0625	16.90	.26	.01	74.0	0.00	.23	0.00	1149.56	114.8	(104.2)
.0625	.1250	15.20	1.84	.19	66.5	0.00	.42	0.00	162.95	103.3	(93.7)
.1250	.2500	34.00	39.50	11.28	148.8	.57	.58	0.00	48.77	550.3	(499.2)
.2500	.5000	30.90	50.34	40.67	135.2	.74	.72	0.00	18.47	751.3	(681.6)
.5000	1.0000	3.00	6.11	7.34	13.1	.83	.84	0.00	11.87	87.1	(79.0)
1.0000	2.0000	0.00	.99	.07	0.0	0.00	.94	0.00	7.64	.5	(0.5)
2.0000	4.0000	0.00	.74	.00	0.0	0.00	1.05	0.00	5.53	.0	
4.0000	8.0000	0.00	.17	0.00	0.0	0.00	1.17	0.00	4.18	0.0	
8.0000	16.0000	0.00	.05	0.00	0.0	0.00	1.29	0.00	3.21	0.0	
TOTALS		100.00	100.00		437.7					1607.3	(1458.2)

The Modified Einstein procedure is quite different from the original Einstein (1950) method. Unlike many formulas for computing sediment transport, it is not a method for predicting sediment transport under future flow conditions. The unique requirement for a discharge measurement and collection of depth-integrated, suspended sediment samples as a base in the computations makes the Modified Einstein procedure serve two main purposes: (1) it gives the unmeasured load to be added to the suspended load, and (2) it provides a check or verification on the most reliable predictive formula. An example of the Modified Einstein computation results is shown in table 4, a printout from the computer program developed by Reclamation. The computer program developed by Reclamation follows the same procedure given in the Bureau of Reclamation (1955) report except for the suspended load exponent or computation of "z" which is described in Bureau of Reclamation (1966) publication.

There are situations where other methods for computing the unmeasured load are needed to either supplement or to replace the Modified Einstein procedure. This usually happens at the higher water discharges when sampling is difficult or with bimodal transport (usually under condition 4 or 5 in table 3) where streambed material is unlike the suspended material.

Several methods or formulas for computing the bedload or total bed material load have been advanced by various investigators over the years. Most of these formulas are based on the principle that the capacity of the stream to transport bed materials varies directly with the differences between the shear stress acting on the bed particles and the critical shear stress required for initiation of particle motion (Herbertson, 1969). One of the better known formulas is that of Einstein (1950), which applied a stochastic approach to sediment transport. Statistical and probability theories are used as a basis for formulas and experimental results are used to establish values for various constants and indexes. Of the various refinements of Einstein's original work, Reclamation has experienced the most success in predicting sediment transport in streams having graded bed material size by use of the Velocity-Xi Adjustment to the Einstein formula as described by Pemberton (1972). Other formulas that are often used to compare with the Modified Einstein method are:

Meyer-Peter, Muller (1948) and by Sheppard (1960)
Schoklitsch, by Shulits (1935)
Ackers and White (1973)
Engelund and Hansen (1967)
Yang (1973)

A description of the theory and development of the above formulas are much beyond the scope of this narrative, and the reader is directed to the listed references for this information.

The recommended approach for extending the range of total sediment loads is to compute total sediment load using the Modified Einstein procedure for as wide a range of discharge as possible and then compare these results to those of the predictive formulas. The one giving results most comparable to the Modified Einstein computations is then used to extend the range to higher discharges. When data are not available for Modified Einstein computations, selection of a predictive formula should be of one which has given good comparative results for streams having similar hydraulic properties and bed material size distributions.

If the bed material is predominately coarse sand greater than about 0.5 mm, gravel-, or cobble-size material, a special sampling program may be used either independently or as a check on the bedload formula. This involves measuring the bedload by a direct measuring sampler such as the Helley-Smith bedload sampler described by Emmett (1980). The sampling procedure can be quite extensive, depending on dunes and irregular streambed patterns. Several samples at 10 to 20 equally spaced verticals in the cross section are necessary to adequately describe the spatial and temporal variations in transport rate.

Once the rate of unmeasured sediment movement has been determined from either the Modified Einstein computations or bedload formulas, an unmeasured load rating curve is drawn. A log-log plot of water discharge versus unmeasured load for these special samples can be analyzed by least squares analysis. A computation of unmeasured load from the correlation of water discharge to unmeasured load is similar to the suspended load computations shown in table 2. Total load is obtained by combining the results of the suspended load and unmeasured load computations.

e. Adjustment to damsite. - Any direct measurement of sediment yield either from reservoir surveys or sediment sampling requires an adjustment in the yield rate from a specific location to that at the damsite. In many cases the sediment yields in acre-feet or tons per square mile (cubic meters or tons per square kilometer) derived from the reservoir survey or at the gaging station can be applied directly to the drainage area above the damsite. If the yield rates are not directly applicable to the drainage area above a damsite, the nine factors shown in table 1 can be used in a calibration technique for adjustment to the damsite.

Reservoir sediment deposition. - Once the estimated sediment inflow to a reservoir has been established, attention must be given to the effect the deposition of this sediment will have upon the life and daily operation of the reservoir. The mean annual sediment inflow, the trap efficiency of the reservoir, the ultimate density of the deposited sediment, and the distribution of the sediment within the reservoir, all must be considered in the design of the dam.

Usually to prevent premature loss of usable storage capacity, an additional volume of storage equal to the anticipated sediment deposition during the economic life of the reservoir is included in the original design. The Bureau of Reclamation requires that provisions be made for sediment storage space whenever the anticipated sediment accumulation during the period of project economic analysis exceeds 5 percent of the total reservoir capacity. A 100-year period of economic analysis and sediment accumulation is typically used for a reservoir; however, less than 100 years of sediment accumulation may be used if the economic analysis would justify a lesser allocation. The allocated sediment space is provided to prevent encroachment on the required conservation storage space for the useful life of the project.

A schematic diagram of anticipated sediment deposition (fig. 8) shows the effect of sediment on storage. A distribution study with 100-year area and capacity curves similar to that shown on the left side of figure 8 is needed whenever the 100-year sediment accumulation is more than 5 percent of the total reservoir capacity. In operational studies of a reservoir for determining the available water supply to satisfy projected water demands over an economic life, an average (50 years for a 100-year economic analysis) can be used of the sediment accumulation during the economic life period. However, the total sediment deposition is used for design purposes to set the sediment elevation at the dam to determine loss of storage due to sediment in any assigned storage space and to be used in determining total storage requirements.

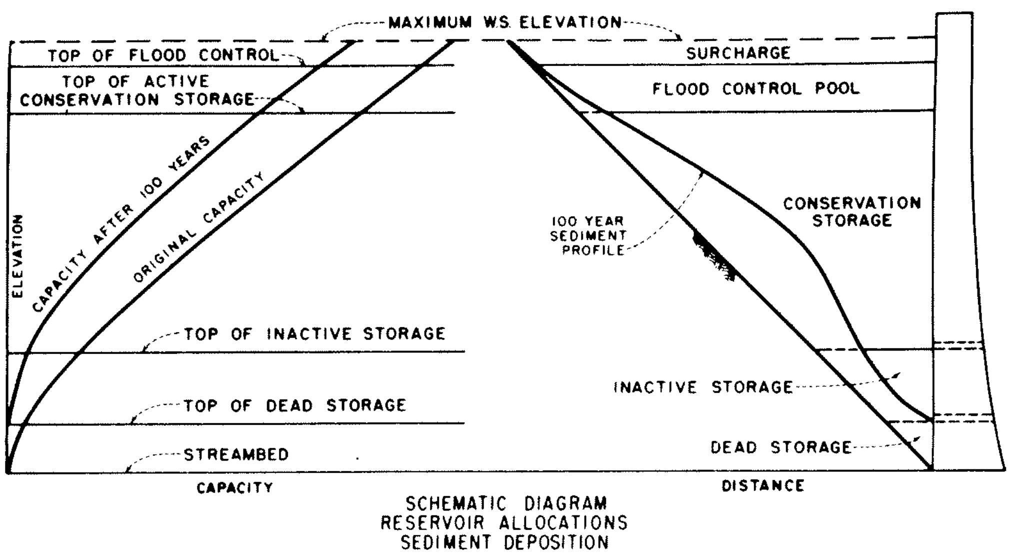


Figure 8. - Schematic diagram, reservoir allocations, sediment deposition.

a. Trap efficiency. - The trap efficiency of a reservoir is defined as the ratio of the quantity of deposited sediment to the total sediment inflow and is dependent primarily upon the sediment particle fall velocity and the rate of flow through the reservoir. Particle fall velocity may be influenced by size and shape of the particle, viscosity of the water, and chemical composition of the water. The rate of flow through the reservoir is determined by the volume of inflow with respect to available storage and the rate of outflow.

Methods for estimating reservoir trap efficiency are empirically based upon measured sediment deposits in a large number of reservoirs. Gunnar Brune (1953) has presented a set of envelope curves for use with normal ponded reservoirs using the capacity-inflow relationship of the reservoirs. The Brune medium curve is reproduced in figure 9.

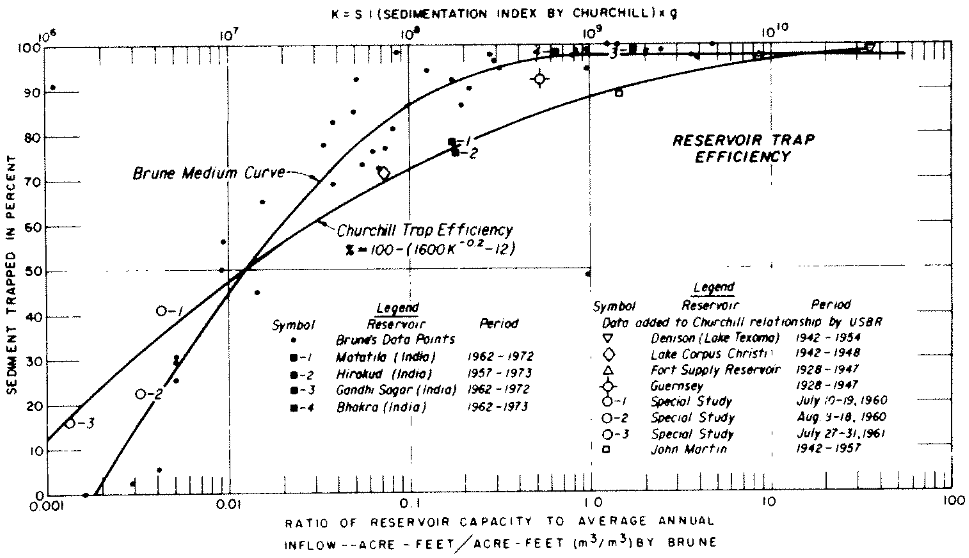


Figure 9. - Trap efficiency curves.

Using data from Tennessee Valley Authority reservoirs, M. A. Churchill (1948) developed a relationship between the percent of incoming sediment passing through a reservoir and the sedimentation index of the reservoir. The sedimentation index is defined as the ratio of the period of retention to the mean velocity through the reservoir. The Churchill curve has been converted to a truly dimensionless expression by multiplying the sedimentation index by g, acceleration due to gravity.

The following description of terms will be helpful in using the Churchill curve:

Capacity. - Capacity of the reservoir in the mean operating pool for the period to be analyzed in cubic feet (cubic meters).

Inflow. - Average daily inflow rate during the study period in cubic feet per second (cubic meters per second).

Period of retention. - Capacity divided by inflow rate.

Length. - Reservoir length in feet (meters) at mean operating pool level.

Velocity. - Mean velocity in feet per second (meters per second), which is arrived at by dividing the inflow by the average cross-sectional area in square feet (square meters). The average cross-sectional area can be determined from the capacity divided by the length.

Sedimentation index. - Period of retention divided by velocity.

Figure 9 provides a good comparison of the Brune and Churchill methods for computing trap efficiencies using techniques developed by Murthy (1980). A general guideline is to use the Brune method for large storage or normal ponded reservoirs and the Churchill curve for settling basins, small reservoirs, flood retarding structures, semidry reservoirs or reservoirs that are continuously sluiced.

When the anticipated sediment accumulation is larger than one-fourth of the reservoir capacity, it is necessary that the trap efficiency be analyzed for incremental periods of the reservoir life. Theoretically, the reservoir trap efficiency will decrease continuously once storage is begun; however, for most reservoirs it is not practical to analyze the trap efficiency in intervals of less than 10 years. The variability of the annual sediment inflow is sufficient reason not to use shorter periods of analysis.

b. Density of deposited sediment. - Samples of deposited sediments in reservoirs have provided useful information on the density of deposits. The density of deposited material in terms of dry mass per unit volume is used to convert total sediment inflow to a reservoir from a mass to a volume. The conversion is necessary when total sediment inflow is computed from a measured suspended and bed material sediment sampling program. Basic factors influencing density of sediment deposits in a reservoir are (1) the manner in which the reservoir is operated, (2) the texture and size of deposited sediment particles, and (3) the compaction or consolidation rate of deposited sediments.

The reservoir operation is probably the most influential of these factors. Sediments that have deposited in reservoirs subjected to considerable drawdown are exposed for long periods and undergo a greater amount of consolidation. Reservoirs operating with a fairly stable pool do not allow the sediment deposits to dry out and consolidate to the same degree.

The size of the incoming sediment particles has a significant effect upon density. Sediment deposits composed of silt and sand will have higher densities than those in which clay predominates. The classification of sediment according to size as proposed by the American Geophysical Union is as follows:

Sediment type	Size range in millimeters
Clay	Less than 0.004
Silt	0.004 to 0.062
Sand	0.062 to 2.0

The accumulation of new sediment deposits, on top of previously deposited sediments, changes the density of earlier deposits. This consolidation affects the average density over the estimated life of the reservoir such as for a 100-year period. A good example on consolidation of deposited sediments is shown in figure 10 taken from the report by Lara and Sanders (1970) for unit weights (densities) in Lake Mead at a sampling location with all clay-size material.

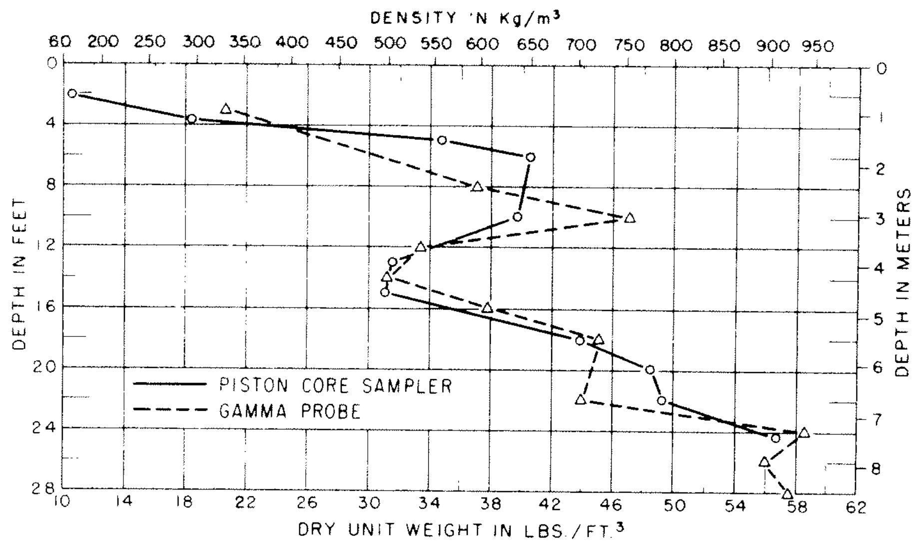


Figure 10. - Comparison of densities on Lake Mead at location 5.

The method that takes into account all three factors in determining the density of deposited sediment is demonstrated in these guidelines. The influence of reservoir operation is most significant because of the amount of consolidation or drying out that can occur in the clay fraction of the deposited material when a reservoir is subjected to considerable drawdown. The size of sediment particles entering the reservoir will also have an effect on density as shown by the variation in initial masses. Some 1,300 samples were statistically analyzed by Lara and Pemberton (1965) for determining mathematical equations of variation of the density of the deposits (sometimes termed unit weight or specific weight) with the type of reservoir operation. Additional data on density of deposited material from reservoir resurveys have supported the Lara and Pemberton (1965) equations (equation 3) which are slightly different than the Lane and Koelzer (1943) equations.

Reservoir operations were classified according to operation as follows:

Operation	Reservoir operation
1	Sediment always submerged or nearly submerged
2	Normally moderate to considerable reservoir drawdown
3	Reservoir normally empty
4	Riverbed sediments

Selection of the proper reservoir operation number usually can be made from the operation study prepared for the reservoir.

Once the reservoir operation number has been selected, the density of the sediment deposits can be estimated using the following equation:

$$W = W_c p_c + W_m p_m + W_s p_s \qquad (3)$$

where

W = unit weight in pounds per cubic foot (density in kilograms per cubic meter)

p_c, p_m, p_s = percentages of clay, silt, and sand, respectively, of the incoming sediment

W_c, W_m, W_s = coefficients of clay, silt, and sand, respectively, which can be obtained from the following tabulation:

Operation	Initial weight (initial mass) in lb/ft^3 (Kg/m^3)		
	W_c	W_m	W_s
1	26 (416)	70 (1120)	97 (1550)
2	35 (561)	71 (1140)	97 (1550)
3	40 (641)	72 (1150)	97 (1550)
4	60 (961)	73 (1170)	97 (1550)

As an example, the following data are known for a proposed reservoir:

Reservoir operation: 1
Size analysis: 23 percent clay, 40 percent silt, and 37 percent sand

then:

$$W = 26\ (0.23) + 70\ (0.40) + 97\ (0.37) = 6.0 + 28.0 + 35.9 = 70\ lb/ft^3\ (1120\ kg/m^3)$$

In determining the density of sediment deposits in reservoirs after a period of reservoir operation it is recognized that part of the sediment will deposit in the reservoir in each of the T years of operation, and each year's deposits will have a different compaction time. Miller (1953) developed an approximation of the integral for determining the average density of all sediment deposited in T years of operation as follows:

$$W_T = W_1 + 0.4343K \left[\frac{T}{T-1} (\log_e T) - 1\right] \qquad (4)$$

where

W_T = average density after T years of reservoir operation
W_1 = initial unit weight (density) as derived from equation 3
K = constant based on type of reservoir operation and sediment size analysis as obtained from the following table:

Reservoir operation	K for inch-pound units (metric units)		
	Sand	Silt	Clay
1	0	5.7 (91)	16 (256)
2	0	1.8 (29)	8.4 (135)
3	0	0 (0)	0 (0)

Using the same example as was used for the initial unit weight (density) computation, the 100-year average values to include compaction are computed as follows:

$$K = 16\ (0.23) + 57\ (0.40) + 0\ (0.37) = 3.68 + 2.28 + 0 = 5.96$$

$$W_{100} = 70 + 0.4343\ (5.96)\left[\frac{100}{99}\ (4.61) - 1\right] = 70 + 2.59\ (3.66) \doteq 79\ \text{lb/ft}^3$$

$$(1270\ \text{kg/m}^3)$$

This value may then be used to convert the initial weights (initial masses) of incoming sediment to the volume it will occupy in the reservoir after 100 years.

c. Sediment distribution within a reservoir. - The data obtained from surveys of existing reservoirs (U.S. Department of Agriculture, 1978) as described in section (b) Reservoir Resurvey Data have been extensively used to develop empirical relationships for predicting sediment distribution patterns in reservoirs. The two most common distribution techniques are illustrated in figures 2 and 3, where sediment is distributed by depth and by longitudinal profile distance, respectively. Both methods clearly show that sediment deposition is not necessarily confined to the lower storage increments of the reservoir.

Sediment accumulations in a reservoir are usually distributed below the top of the conservation pool or normal water surface. However, if the reservoir has a flood control pool and it is anticipated that the water surface will be held within this pool for significant periods of time, a portion of the sediment accumulation may be deposited within this pool. Figure 11 is a plot of data from 11 Great Plains reservoirs in the United States which may be used as a guide in estimating the portion of the total sediment accumulation which will deposit above the normal water surface. This plot should

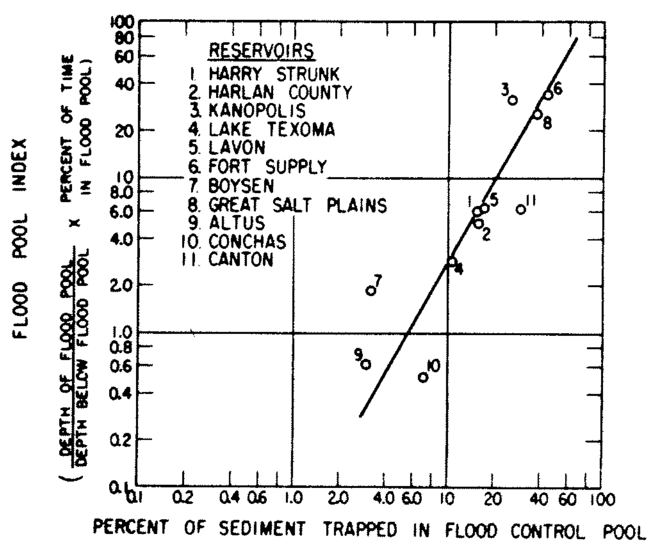


Figure 11. - Sediment deposited in flood control pool.

be regarded as a rough guide only, and the estimate obtained from it should be tempered with some judgment based upon the proposed reservoir operation and the nature of the incoming sediment. This curve is based on a limited amount of data and may be revised as more information becomes available.

The term flood pool index is computed as the ratio of the flood control pool depth to the depth below the pool, multiplied by the percent of time the reservoir water surface will be within the flood control pool. This information for a proposed reservoir must be obtained from the reservoir operation study.

Once the quantity of sediment which will deposit below the normal water surface has been established, the Empirical Area-Reduction Method may be used to estimate the distribution. This method was first developed from data gathered in the resurvey of 30 reservoirs and is described by Borland and Miller (1960) with revisions by Lara (1962). The method recognizes that distribution of sediment is dependent upon (1) the manner in which the reservoir is to be operated, (2) the texture and size of deposited sediment particle, (3) shape of the reservoir, and (4) volume of sediment deposited in the reservoir. However, the shape factor was adopted as the major criteria for development of empirically derived design curves for use in distributing sediment. The shape of the reservoir is defined by the depth to capacity relationship where "m" is the reciprocal of the slope of the depth versus capacity plot on a logarithmic paper. The classification of reservoirs on this basis is as follows:

Reservoir type	Classification	m
I	Lake	3.5 to 4.5
II	Flood plain-foothill	2.5 to 3.5
III	Hill	1.5 to 2.5
IV	Normally empty	

The procedure now used by Reclamation for distribution with depth is that of using design curves shown in figure 12. With equal weight applied to reservoir operation and shape, a type distribution is selected from table 5. In those cases where a choice of two types are given, then a judicious decision can be made on whether the reservoir operation or shape of reservoir is more influential. The texture and size of deposited sediments could be considered in this judgment analysis from the following guidelines:

Predominant size	Type
Sand or coarser	I
Silt	II
Clay	III

The size of sediments in most river systems is a mixture of clay, silt, and sand and has been found to be least important in selecting the Design Type Curve from figure 12. Only for those cases with two possible type distributions should size of sediment be considered in selecting the Design Type Curve.

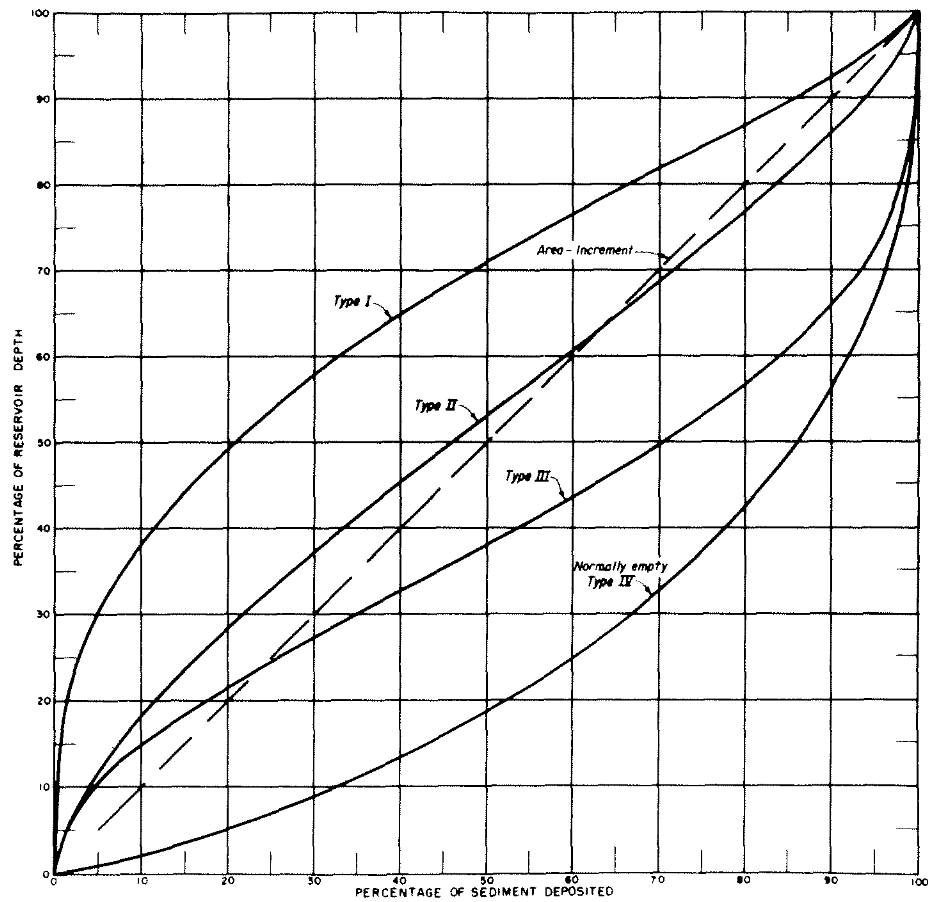


Figure 12. - Sediment distribution design curves.

Table 5. - Design type curve selection

Reservoir operation: Class	Reservoir operation: Type	Shape: Class	Shape: Type	Weighted type
Sediment submerged	I	Lake	I	I
		Flood plain - foothill	II	I or II
		Hill and gorge	III	II
Moderate drawdown	II	Lake	I	I or II
		Flood plain - foothill	II	II
		Hill and gorge	III	II or III
Considerable drawdown	III	Lake	I	II
		Flood plain - foothill	II	II or III
		Hill and gorge	III	III
Normally empty	IV	All shapes		IV

The Lara publication (1962) provides the detail on distributing sediment in a reservoir by the Empirical-Area Reduction Method. The appropriate design type curve is selected using the weighting procedure shown in table 5. A computer program written by Hudspeth and Trietsch (1978) can be also used for distributing sediment by either the Empirical Area-Reduction Method or the Area-Increment Method. The Area-Increment Method is based on the assumption that the area of sediment deposition remains constant throughout the reservoir depth. It is almost identical to the type II design curve and is often used to estimate the new zero capacity elevation at the dam.

An example of a sediment distribution study is given for Theodore Roosevelt Dam located on the Salt River in Arizona. Construction of the dam was completed in 1909 and a complete survey of the reservoir made in 1981. The reservoir had an original total capacity of 1 530 500 acre-feet (188 800^6 m^3) at elevation 2136 feet (651.0 m), the top of the active conservation pool. The purpose of this example is to (1) compare the actual survey of 1981 with the distribution procedures, (2) show all of the steps involved in a distribution study, and (3) provide changes in capacity and projected sediment depths at the dam for 100, 200, and 300 years.

Table 6 gives the pertinent area-capacity data necessary to evaluate the actual 1981 survey and for use as a base in the distribution study. The total sediment accumulation in Theodore Roosevelt Lake as determined from the 1981 survey was 193 765 acre-feet (239 x 10^6 m^3). In the 72.4 years from closure of the dam in May 1909, until the survey in September 1981, the average annual sediment deposited was 2676 acre-feet (3301 x 10^6 m^3) per year. The survey data from table 6 were used to draw the sediment distribution design curve on figure 13. To check the most appropriate design curve by the Empirical Area-Reduction Method, the volume of sediment accumulated in Theodore Roosevelt Lake from 1909 to 1981 was distributed by both a type II and III distribution, as shown in figure 13. This comparison indicates that type II more closely resembles the actual survey. A plot of the area and capacity data from table 6 is shown on figure 14.

The first step in the distribution study for the 100-, 200-, and 300-year period is a determination of the rate of sediment accumulation. In the case of Theodore Roosevelt Lake, the rate determined from the 1981 survey used for future projections with the assumption that the compaction or density of deposits will not change is:

Years	Sediment volume: Acre-feet	Sediment volume: (10^3 m^3)
72.4 (1981)	193 765	239 009
100	267 600	330 100
200	535 200	660 200
300	802 800	990 300

Table 6. - Reservoir area and capacity data
Theodore Roosevelt Lake

Elevation		Original (1909)				Actual survey (1981)			
		Area		Capacity		Area		Capacity	
Feet	Meters	Acres	Hectare	10^3 acre-ft	10^6 m^3	Acres	Hectare	10^3 acre-ft	10^6 m^3
2136	651.0	17 785	7 198	1 530.5	1 888	17 337	7 016	1 336.7	1 649
2130	649.2	17 203	6 962	1 425.5	1 758	16 670	6 783	1 234.3	1 523
2120	646.2	16 177	6 547	1 258.5	1 552	15 617	6 320	1 072.4	1 323
2110	643.1	15 095	6 109	1 102.2	1 360	14 441	5 844	922.3	1 138
2100	640.1	14 104	5 708	956.5	1 180	13 555	5 486	782.6	965
2090	637.0	13 247	5 361	819.3	1 011	12 746	5 158	650.5	802
2080	634.0	11 939	4 832	693.3	855	11 331	4 586	530.0	654
2070	630.9	10 638	4 305	580.6	716	9 842	3 983	424.0	523
2060	627.9	9 482	3 837	479.9	592	8 230	3 331	333.8	412
2050	624.8	8 262	3 344	391.2	483	6 781	2 744	258.9	319
2040	621.8	7 106	2 876	314.6	388	5 569	2 254	197.6	244
2030	618.7	6 216	2 516	248.0	306	4 847	1 962	145.6	180
2020	615.7	5 286	2 139	190.3	235	4 212	1 705	100.3	124
2010	612.6	4 264	1 726	142.9	176	3 387	1 371	61.6	76.0
2000	609.6	3 544	1 434	103.8	128	2 036	824	35.0	43.2
1990	606.6	2 744	1 110	72.3	89.2	1 304	528	18.7	23.0
1980	603.5	1 985	803	48.9	60.3	903	365	7.6	9.4
1970	600.5	1 428	578	31.9	39.4	382	155	0.8	1.0
1960	597.4	1 020	413	19.7	24.4	1/ 0	1/ 0	1/ 0	1/ 0
1950	594.4	677	274	11.3	14.0				
1940	591.3	419	170	5.9	7.3				
1930	588.3	227	91.9	2.7	3.4				
1920	585.2	117	47.3	1.1	1.3				
1910	582.2	52	21.0	0.2	0.3				
1902	579.7	0	0	0	0				

1/ Sediment elevation at dam for 1981 survey is 1966 feet (599.2 m).

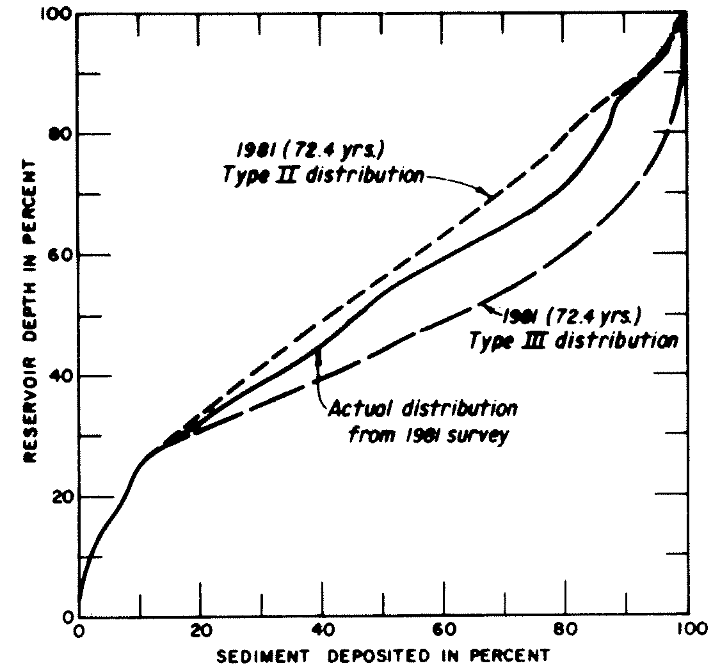


Figure 13. - Sediment distribution for Theodore Roosevelt Lake.

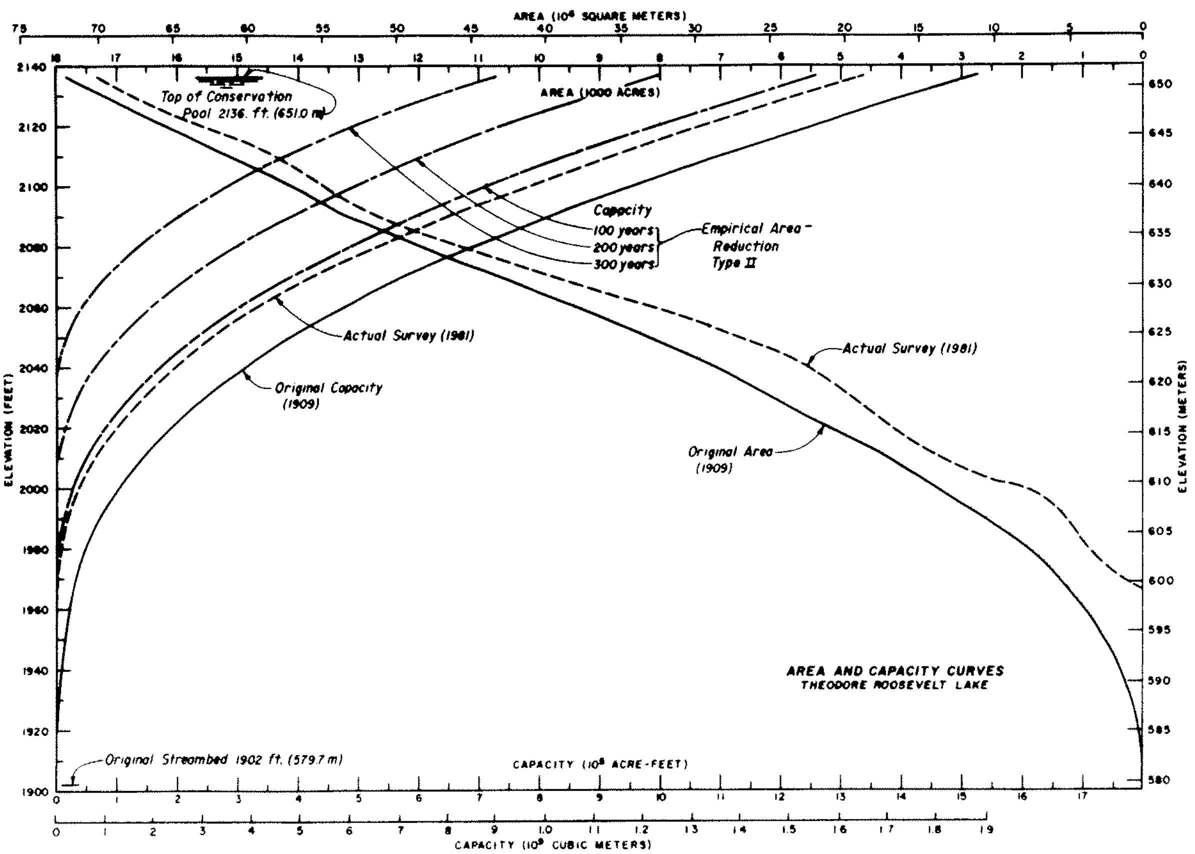


Figure 14. - Area and capacity curves for Theodore Roosevelt Lake.

There were no data on trap efficiency to apply to the above projections. The use of the rate from the 1981 survey results assumes that the trap efficiency for the first 72.4 years will remain the same through 300 years. In cases where sediment accumulation is determined from the total sediment load at a gaging station, then trap efficiency by use of figure 9 and densities from equations 3 and 4 are needed for computing the volume of sediment accumulation.

To complete this example a logarithmic plot of the depth-capacity relationship for the original (1909) survey (fig. 15) for Theodore Roosevelt Lake, provided the shape factor for type classification. Although the lower portion of the reservoir falls slightly in the type III, the upper portion and overall slope indicates a type II classification. When assigning a type classification for either an existing reservoir or in distributing sediment on top of previous sediment deposits that the stage-capacity relationship only be plotted for the original survey. Studies have shown that a reservoir does not change type with continued sediment depositions. Once a reservoir has been assigned a type by shape, this classification will not change. However, it is possible that a change in reservoir operation could produce a new weighted type, see table 5.

The next step in the distribution study is computation of the elevation of sediment deposited at the dam. A set of computations for determining the depth of sediment at the dam is shown in table 7. The relative depth and a dimensionless function from the original area and capacity curves for Theodore Roosevelt Lake are computed as shown in table 7 with the function:

$$F = \frac{S-V_h}{H \cdot A_h} \qquad (5)$$

where

F = dimensionless function of total sediment deposition, capacity, depth, and area
S = total sediment deposition
V_h = reservoir capacity at a given elevation h
H = original depth of reservoir
A_h = reservoir area at a given elevation h

A plot of the data points from table 7 is superimposed on figure 16 and the p value (relative depth) at which the line for any year crosses; the appropriate type curve will give the relative depth p_o, equal to the new zero elevation at the dam. Figure 16 contains plotted curves of the full range of F values for all four reservoir types and the Area-Increment Method as developed from the capacity and area design curves. For Theodore Roosevelt Dam, the intersect points for type II as well as for the Area-Increment Method curves gave sediment depths shown in table 8. The Area-Increment Method is often selected because it will always intersect the F curve and, in many cases, gives a good check on the new zero capacity elevation at the dam. In the case of Theodore Roosevelt Dam, the 1981

Table 7. - Determination of elevation of sediment at Theodore Roosevelt Dam

	Year	Total sediment deposition	Original depth of reservoir
1981 survey	72.4	193 765 acre-ft (239 10^6 m^3)	234 feet (71.3 m)
	100	267 600 acre-ft (330 10^6 m^3)	
	200	535 200 acre-ft (660 10^6 m^3)	
	300	802 800 acre-ft (990 10^6 m^3)	

Elevation		P relative depth	Original survey (1909)						72.4 years			100 years			200 years			300 years		
			V_h capacity		A_h area		$H \cdot A_h$ 10^6		$S-V_h$		$\frac{F}{S-V_h}$	$S-V_h$		$\frac{F}{S-V_h}$	$S-V_h$		$\frac{F}{S-V_h}$	$S-V_h$		$\frac{F}{S-V_h}$
ft	10 m		acre-ft	10^6 m^3	acres	10^6 m^2	acre-ft	10^6 m^3	acre-ft	10^6 m^3	$H \cdot A_h$	acre-ft	10^6 m^3	$H \cdot A_h$	acre-ft	10^6 m^3	$H \cdot A_h$	acre-ft	10^6 m^3	$H \cdot A_h$
2080	6340	0.761	693 315	855	11 939	48.3	2.79	3 440										109 485	135	0.0392
2070	6309	0.718	580 590	716	10 638	43.1	2.49	3 070										222 210	274	0.0892
2060	6279	0.675	479 928	592	9 482	38.4	2.22	2 700							55 272	68.2	0.0249	322 872	398	0.145
2050	6248	0.632	391 207	483	8 262	33.4	1.93	2 380							143 993	178	0.0746	411 593	508	0.213
2040	6218	0.590	314 623	388	7 106	28.8	1.66	2 050							220 577	272	0.133	488 177	602	0.294
2030	6187	0.547	248 009	306	6 216	25.2	1.45	1 800				19 591	24.2	0.0135	287 191	354	0.198	554 791	684	0.383
2020	6157	0.504	190 334	235	5 286	21.4	1.24	1 530				77 266	95.3	0.0623	344 866	425	0.278	612 466	755	0.494
2010	6126	0.462	142 903	176	4 264	17.3	0.998	1 230	50 862	62.7	0.0510	124 697	154	0.125	392 297	484	0.393	659 897	814	0.661
2000	6096	0.419	103 787	128	3 544	14.3	0.829	1 020	89 978	111	0.109	163 813	202	0.198	431 413	532	0.520	699 013	862	0.843
1990	6066	0.376	72 347	89.2	2 744	11.1	0.642	791	121 418	149.8	0.189	195 253	241	0.304	462 853	571	0.721	730 453	901	1.138
1980	6035	0.333	48 867	60.3	1 985	8.03	0.464	573	144 898	178.7	0.312	218 733	270	0.471	486 333	600	1.048	753 933	930	1.625
1970	6005	0.291	31 935	39.4	1 428	5.78	0.334	412	161 830	199.6	0.485	235 665	291	0.706	503 265	621	1.507	770 865	951	2.308
1960	5974	0.248	19 743	24.4	1 020	4.13	0.239	294	174 022	214.6	0.730	247 857	306	1.037	515 457	636	2.157	783 057	966	3.276
1950	5944	0.205	11 328	14.0	677	2.74	0.158	195	182 437	225	1.155	256 272	316	1.622	523 872	646	3.316	791 472	976	5.009

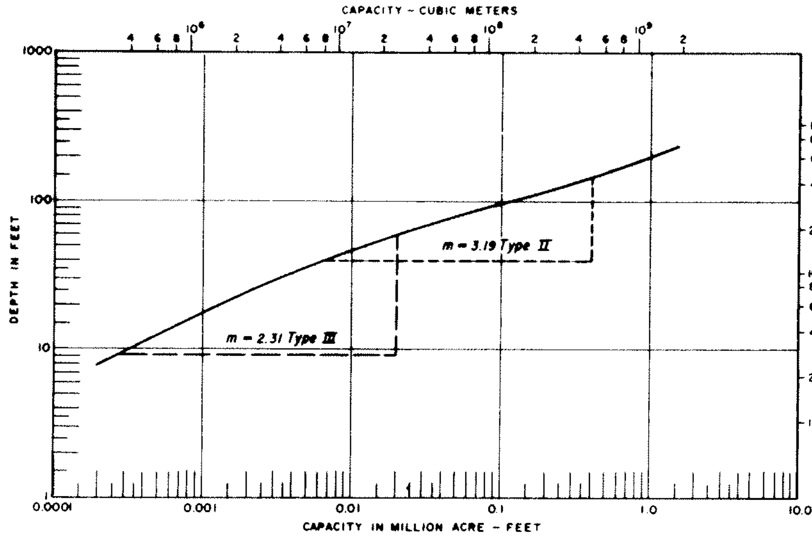


Figure 15. - Depth versus capacity for Theodore Roosevelt Lake.

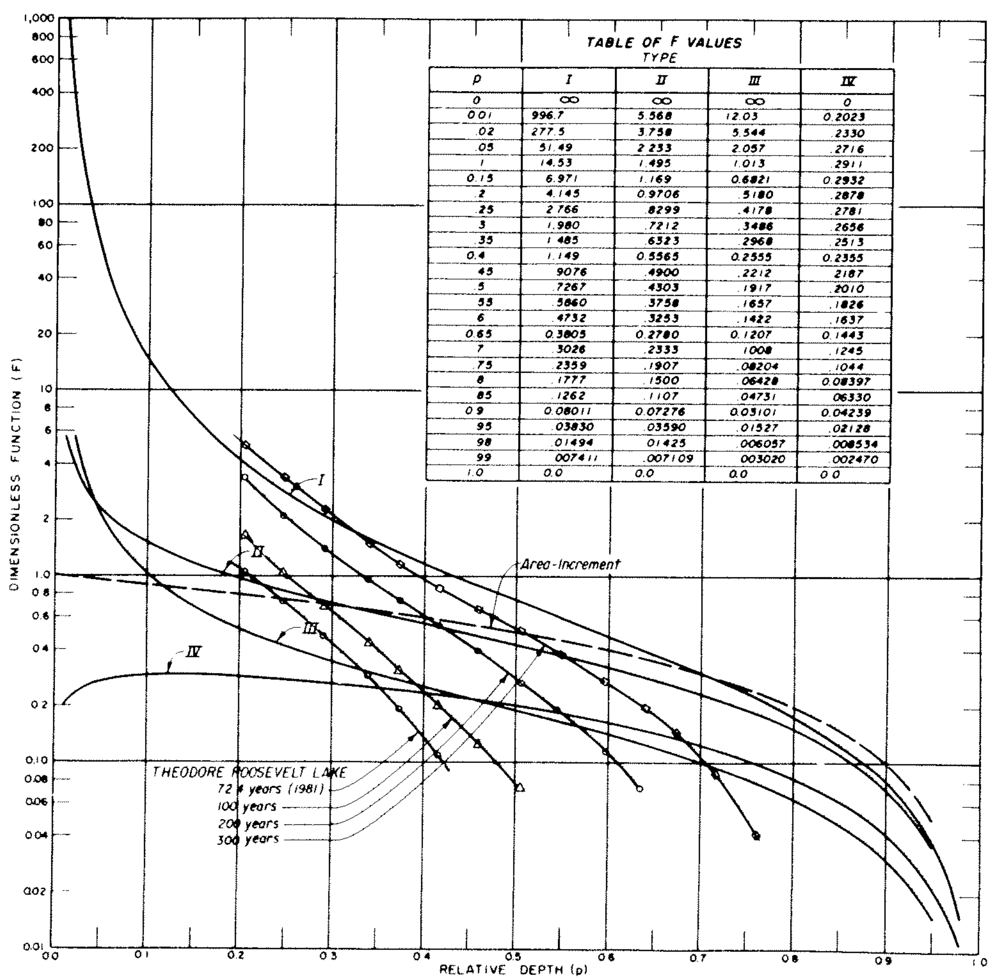


TABLE OF F VALUES

p	Type I	Type II	Type III	Type IV
0	∞	∞	∞	0
0.01	996.7	5.568	12.03	0.2023
.02	277.5	3.758	5.544	.2330
.05	51.49	2.233	2.057	.2716
.1	14.53	1.495	1.013	.2911
0.15	6.971	1.169	0.6821	0.2932
.2	4.145	0.9706	.5180	.2878
.25	2.766	.8299	.4178	.2781
.3	1.980	.7212	.3486	.2656
.35	1.485	.6323	.2968	.2513
0.4	1.149	0.5565	0.2555	0.2355
.45	.9076	.4900	.2212	.2187
.5	.7267	.4303	.1917	.2010
.55	.5860	.3758	.1657	.1826
.6	.4732	.3253	.1422	.1637
0.65	0.3805	0.2780	0.1207	0.1443
.7	.3026	.2333	.1008	.1245
.75	.2359	.1907	.08204	.1044
.8	.1777	.1500	.06428	0.08397
.85	.1262	.1107	.04731	.06330
0.9	0.08011	0.07276	0.03101	0.04239
.95	.03830	.03590	.01527	.02128
.98	.01494	.01425	.006057	.008534
.99	.007411	.007109	.003020	.002470
1.0	0.0	0.0	0.0	0.0

Figure 16. - Curves to determine the depth of sediment at the dam.

survey had an observed elevation at the dam of 1966 feet (599.2 m) which was in better agreement with the Area-Increment Method value than any of the type curves. Data from table 8 can be used to predict useful life of a reservoir or projection beyond the 300 years.

Table 8. - Elevation of sediment at Theodore Roosevelt Dam
H = 234 feet (71.3 m)

Years	Type II				Area increment			
			Elevation				Elevation	
	p_o	p_oH	feet	meters	p_o	p_oH	feet	meters
72.4 (1981)	0.23	54	1956	596.2	0.247	58	1960	597.4
100	0.284	66	1968	599.8	0.290	68	1970	600.5
200	0.418	98	2000	609.6	0.4	94	1996	608.4
300	0.553	129	2031	619.0	0.506	118	2020	615.7

The final step in the distribution study is to distribute a specified volume of sediment which for the example selected involved the 72.4-, 100-, 200-, and 300-year volume in Theodore Roosevelt Lake by the type II design curve. The results of this distribution using procedures described by Lara (1962) or the computer program by Hudspeth and Trietsch (1978) are shown in figure 14. An example of the computer results for the 100-year distribution by use of the Empirical Area-Reduction Method and type II design curves is shown in table 9. Although the example given is for type II, the equations for the relative sediment area, a, for each type follows:

Type	Equation	
I	$a = 5.074\ p^{1.85}\ (1-p)^{0.35}$	(6)
II	$a = 2.487\ p^{0.57}\ (1-p)^{0.41}$	(7)
III	$a = 16.967\ p^{1.15}\ (1-p)^{2.32}$	(8)
IV	$a = 1.486\ p^{-0.25}\ (1-p)^{1.34}$	(9)

where

a = relative sediment area
p = relative depth of reservoir measured from the bottom
p_o = relative depth at zero capacity

d. Delta deposits. - Another phenomenon of reservoir sediment deposition is the distribution of sediment longitudinally as illustrated in figure 3 for Lake Mead. The extreme upstream portion of the deposition profile

Table 9. - Theodore Roosevelt Lake - Type II reservoir sediment deposition study - empirical area reduction method-sediment inflow, 267 600 acre-feet (inch-pound units)

Elev. (ft)	Original Area (acre)	Original Capacity (acre-ft)	Relative Depth	Relative Area	Sediment Area (acre)	Sediment Volume (acre-ft)	Revised Area (acre)	Revised Capacity (ft)
2136.0	17 785.0	1 530 499	1.000	0.000	0.0	267 600	17 785.0	1 262 899
2130.0	17 203.0	1 425 512	0.974	0.546	699.1	265 503	16 503.9	1 160 009
2120.0	16 177.0	1 258 547	0.932	0.795	1018.8	256 914	15 158.2	1 001 633
2110.0	15 095.0	1 102 215	0.889	0.945	1210.3	245 768	13 884.7	856 447
2100.0	14 104.0	956 455	0.846	1.050	1344.8	232 993	12 759.2	723 462
2090.0	13 247.0	819 272	0.803	1.127	1443.6	219 051	11 803.4	600 221
2080.0	11 939.0	693 315	0.761	1.184	1516.9	204 248	10 422.1	489 067
2070.0	10 638.0	580 590	0.718	1.225	1570.0	188 814	9 068.0	391 776
2060.0	9 422.0	479 928	0.675	1.254	1606.3	172 293	7 875.7	306 996
2050.0	8 262.0	391 207	0.632	1.271	1628.0	156 761	6 634.0	234 446
2040.0	7 106.0	314 623	0.590	1.277	1636.5	140 438	5 469.5	174 185
2030.0	6 216.0	248 009	0.547	1.274	1632.8	124 092	4 583.2	123 917
2020.0	5 286.0	190 334	0.504	1.263	1617.6	107 840	3 668.4	82 494
2010.0	4 264.0	142 903	0.462	1.242	1591.0	91 797	2 673.0	51 106
2000.0	3 544.0	103 787	0.419	1.212	1553.1	76 076	1 990.9	27 711
1990.0	2 744.0	72 347	0.376	1.174	1503.8	60 792	1 240.2	11 555
1980.0	1 985.0	48 867	0.333	1.126	1443.0	46 057	542.0	2 810
1970.0	1 428.0	31 935	0.291	1.068	1381.5	31 935	46.5	33
1968.6	1 369.7	29 983	0.284	1.059	1369.7	29 983	0.0	0
1960.0	1 020.0	19 743	0.248	0.999	1020.0	19 743	0.0	0
1950.0	677.0	11 328	0.205	0.918	677.0	11 328	0.0	0
1940.0	419.0	5 893	0.162	0.821	419.0	5 893	0.0	0
1930.0	227.0	2 735	0.120	0.704	227.0	2 735	0.0	0
1920.0	117.0	1 059	0.077	0.558	117.0	1 059	0.0	0
1910.0	52.0	211	0.034	0.358	52.0	211	0.0	0
1902.0	0.0	0	0.000	0.000	0.0	0	0.0	0

Table 9. - Theodore Roosevelt Lake - Type II reservoir sediment deposition study - empirical area reduction method-sediment inflow, 330 085 dam^3 (metric units) - continued

Elev. (m)	Original		Relative		Sediment		Revised	
	Area (ha)	Capacity (dam^3)	Depth	Area	Area (ha)	Volume (dam^3)	Area (ha)	Capacity (dam^3)
651.05	7197.6	1 887 871	1.000	0.000	0.0	330 085	7197.6	1 557 786
649.22	6962.1	1 758 369	0.974	0.546	282.9	327 498	6679.1	1 430 871
646.18	6546.8	1 552 418	0.932	0.795	412.3	316 903	6134.5	1 235 515
643.13	6108.9	1 359 582	0.889	0.945	489.8	303 155	5619.1	1 056 427
640.08	5707.9	1 179 787	0.846	1.059	544.2	287 397	5163.7	892 391
637.03	5361.1	1 010 572	0.803	1.127	534.2	270 199	4776.8	740 373
633.98	4831.7	855 204	0.761	1.184	613.9	251 940	4217.8	603 264
630.94	4305.2	716 158	0.718	1.225	635.4	232 901	3669.8	483 256
627.89	3837.4	591 991	0.675	1.254	650.1	213 311	3187.3	378 680
624.84	3343.6	482 554	0.632	1.271	658.8	193 364	2684.8	289 190
621.79	2875.8	388 087	0.590	1.277	662.3	173 230	2213.5	214 858
618.74	2515.6	305 919	0.547	1.274	660.8	153 066	1854.8	152 853
615.70	2139.2	234 777	0.504	1.263	654.6	133 019	1484.6	101 758
612.65	1725.6	176 271	0.462	1.242	643.9	113 230	1081.8	63 041
609.60	1434.3	128 021	0.419	1.212	628.5	93 839	805.7	34 183
606.55	1110.5	89 249	0.376	1.174	608.6	74 985	501.9	14 255
603.50	803.3	60 277	0.333	1.126	584.0	56 810	219.4	3 468
600.46	577.9	39 392	0.291	1.068	559.0	39 392	19.0	42
600.92	554.1	36 967	0.284	1.059	554.1	36 967	0.0	0
597.41	412.8	24 353	0.248	0.999	412.8	24 353	0.0	0
594.36	274.0	13 973	0.205	0.918	274.0	13 973	0.0	0
591.31	169.6	7 269	0.162	0.821	169.6	7 269	0.0	0
588.26	91.9	3 374	0.120	0.704	91.9	3 374	0.0	0
585.22	47.3	1 306	0.077	0.558	47.3	1 306	0.0	0
582.17	21.0	260	0.034	0.358	21.0	260	0.0	0
579.73	0.0	0	0.000	0.000	0.0	0	0.0	0

is the formation of delta deposits. The major consequence of these delta deposits is the raising of the backwater elevations in the channel upstream from a reservoir. Therefore, the delta may cause a flood potential that would not be anticipated from preproject channel conditions and proposed reservoir operating water surfaces. Predicting the delta development within a reservoir is a complex problem because of the variables such as operation of the reservoir, sizes of sediment, and hydraulics (in particular, the width of the upper reaches of the reservoir). Sediments deposited in the delta are continually being reworked into the downstream storage area at times of low reservoir stage and during extreme flood discharges.

A delta study is needed for situations involving the construction of railroad or highway bridges in the delta area, defining inundated property such as urban or farmland, and design of protective structures to control inundation of property. The two phases of the delta study are to physically locate the delta and then with the delta in place to run backwater computations through the upstream channel for defining lands that would be inundated due to a downstream reservoir and delta. The 100-year flood peak discharge is often used for inundation comparison in the flood plain, with a 50-year delta to represent average conditions for the 100-year event. If structures such as bridges or levees to protect homes are being designed in the headwater area, then the delta should represent 100 years of sediment deposits to sustain no damage for at least a 100-year period.

The prediction of delta formation is as yet an empirical procedure based upon observed delta deposits in existing reservoirs. A typical delta profile is shown on figure 17. It is defined by a topset slope, foreset slope, and a pivot point between the two slopes at the median or 50 percent reservoir operating level. The quantity of material to be placed in the delta is assumed to be equal to the volume of sand-size material or coarser (>0.062 mm) entering the reservoir for the 50- or 100-year period. A trial and error method, utilizing survey or topographic data and volume computations by average end-area method, is used to arrive at a final delta location.

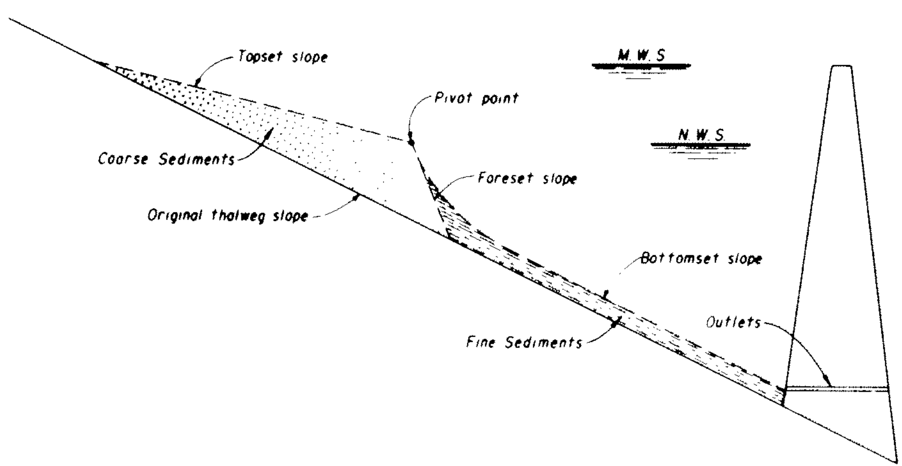


Figure 17. - Typical sediment deposition profile.

The topset slope of the delta is computed by one or more of several methods: (1) a statistical analysis of existing delta slopes which supports a value equal to one-half of the existing channel slope (fig. 18), (2) topset slope from a comparable existing reservoir, or (3) zero bedload transport slope from a bedload equations such as the Meyer-Peter, Muller (1948) and by Sheppard (1960) or Schoklitsch by Shulits (1935). An example of the topset slope computed by the Meyer-Peter, Muller beginning transport equation for zero bedload transport is given by:

$$S = K \frac{\frac{Q}{Q_B}\left(\frac{n_s}{D_{90}^{1/6}}\right)^{3/2}}{d} D \qquad (10)$$

where

S = topset slope
K = coefficient equal to 0.19 (inch-pound units) or 0.058 (metric units)
$\frac{Q}{Q_B}$ = ratio of total flow in ft^3/s (m^3/s) to flow over bed of stream in ft^3/s (m^3/s). Discharge is referred to as dominant discharge and is usually determined by either channel bank full flow or as the mean annual flood peak.
D = diameter of bed material on topset slope usually determined as weighted mean diameter in millimeters
D_{90} = diameter of bed material for 90 percent finer than in millimeters
d = maximum channel depth at dominant discharge in feet (m)
n_s = Mannings roughness coefficient for the bed of channel sometimes computed as $D_{90}^{1/6}/26$.

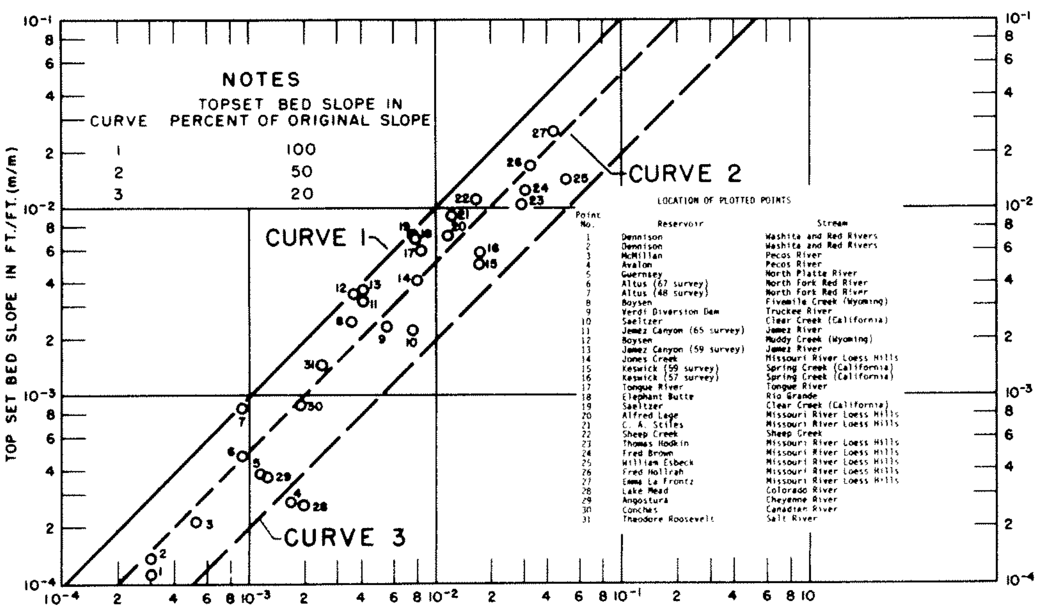


Figure 18. - Topset slope versus original stream slope from existing reservoirs.

The Meyer-Peter, Muller equation or any other equation selected for zero transport will yield a slope at which the bed material will no longer be transported, which must necessarily be true for the delta to form.

The location of the pivot point between the topset and foreset slopes depends primarily on the operation of the reservoir and the existing channel slope in the delta area. If the reservoir is operated near the top of the conservation pool a large portion of the time, the elevation of the top of the conservation pool will be the pivot point elevation. Conversely, if the reservoir water surface has frequent fluctuations and a deeply entrenched inflow channel, a mean operating pool elevation should be used to establish the pivot point. In the extreme situation when a reservoir is emptied every year during the floodpeak flows for sluicing sediment, there will be no pivot point.

The upstream end of the delta is set at the intersection of the maximum water surface and the original streambed, and the topset slope is projected from that point to the anticipated pivot point elevation to begin the first trial computations of delta volume.

The average of foreset slopes observed in Bureau of Reclamation reservoir resurveys is 6.5 times the topset slope. However, some reservoirs exhibit a foreset slope considerably greater than this; i.e., Lake Mead foreset slope is 100 times the topset. By adopting a foreset slope of 6.5 times the topset, the first trial delta fit can be completed.

The volume of sediment computed from the channel cross sections with the delta imposed on them should agree with the volume of sand size or larger material anticipated to come from the delta stream. The quantity of sediment in the delta above normal water surface elevation should also agree with that estimated to deposit above the normal operating level as shown in figure 11. If the adjustment necessary to attain agreement is minor, it can usually be accomplished by a small change in the foreset slope. If a significant change in delta size is needed, the pivot point can be moved forward or backward in the reservoir while maintaining the previously determined elevation of the point. The topset slope is then projected backward from the new pivot point location and the delta volume is again computed. The intersection of the delta topset and the original streambed may fall above the maximum water surface elevation, a condition that has been observed in small reservoirs.

Downstream channel effects. -

a. General degradation. - The trapping of sediment in a reservoir accompanied with clear water releases from the dam upsets the regime or state of quasi-equilibrium of the downstream river channel. A natural flowing stream transporting sediment is usually in equilibrium or regime (Lane, 1955) with no long-term trend toward aggradation or degradation. The release of clear water either through the outlets, powerplant, or spillway will upset this natural stable condition with degradation of the channel bed and banks. The degradation process moves progressively downstream until it reaches a point where the sediment being transported results in a stable channel or equilibrium. Some reservoirs that have lower trap efficiencies may release water with colloidal clay material <0.004 mm, but then releases will usually have a minor influence on retarding the downstream degradation. The one exception to a clear water release would be a reservoir that has planned sluicing with low-level outlets that have capacity equal to the high river discharges for moving large amounts of sediment into the downstream channel. Any sediment sluiced through a dam, especially of sand-size material >0.062 mm, would reduce the expected downstream channel degradation.

The techniques for computing degradation below a dam can vary considerably depending on size of sediments in the bed and banks, release discharges at the dam, and sophistication desired in results. Sophisticated mathematical modeling solutions (Corps of Engineers, 1977) for computing degradation by computer application are becoming available such as the model being developed for Reclamation which is scheduled for completion in 1983. These models simulate the behavior of an alluvial channel by combining a steady-state backwater computation for defining channel hydraulics with a sediment transport model. Through the use of the electronic computer, flows can be simulated over any selected time frame to reflect continual changes in both water surface and the corresponding bed surface profiles to span a 50- to 100-year period. The models, still undergoing development, are being used on many river channels but are considered more applicable to some of the large more uniform width-depth-type river channels in the United States such as the Missouri, Sacramento, or Mississippi Rivers.

Until the mathematical models prove adaptable to meet all river conditions, Reclamation's approach to degradation below dams is to apply either a stable slope or an armoring analyses. Both of these two distinct approaches for estimating the depth or amount of degradation that will occur downstream from a dam or similar structure are dependent on the type of material forming the bed of the river channel described by Pemberton and Lara (1982).

In cases where the streambed is composed of transportable material and the material extends to depths greater than that to which the channel can be expected to degrade, the approach most useful is that of computing the stable channel slope or limiting slope, estimating the volume of expected degradation, and then determining a three-slope channel profile which fits these values. However, if large size or coarse material which cannot be transported by normal river discharges exists in sufficient quantities, an armor layer will develop as the finer material is sorted out and transported downstream. Vertical degradation will proceed at a progressively slower rate until the armor is of sufficient depth to inhibit further degradation.

b. Armoring method. - A less detailed procedure, which should be tested first for computing degradation below a dam, is the armoring control method. This is especially applicable if large size or coarse material exists in the channel bottom that cannot be transported by normal river discharge and is in sufficient quantity to provide an armor layer as described by Pemberton (1976) below Glen Canyon Dam on the Colorado River. Under the armoring process, the finer transportable material is sorted out, and vertical degradation proceeds at a progressively slower rate until armor is of sufficient depth to control further degradation. An armoring layer can usually be anticipated if there is approximately 10 percent or more of the bed material of armoring size or larger. The armoring computations assume that an armoring layer will form as shown in figure 19 as follows:

$$y_a = y - y_d \tag{11}$$

where

y_a = thickness of armoring layer
y = depth from original streambed to bottom of the armoring layer
y_d = depth from the original streambed to top of armoring layer or the depth of degradation and by definition

$$y_a = (\Delta p)_y \tag{12}$$

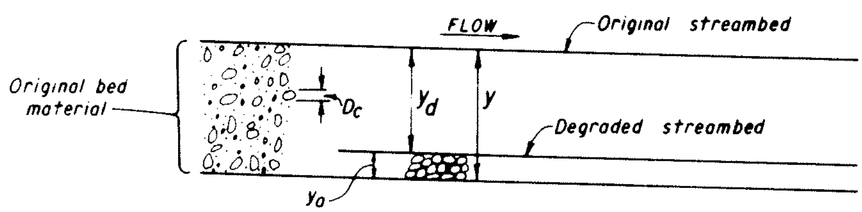
where

Δp = decimal percentage of material larger than the armoring size

The two equations are combined to:

$$y_d = y_a \left(\frac{1}{\Delta p} - 1\right) \tag{13}$$

The depth, y_a, to armor will vary with size of particle needed but is usually assumed to vary by three armoring particle diameters or 0.5 foot (0.15 meter), whichever is smaller for use in design. Although armoring has been observed to occur with less than three particle diameters, variability of channel bed material and occurrence of peak design discharges dictate the use of a thicker armor layer.



y = Depth to bottom of the armoring layer

y_d = Depth of degradation

y_a = Armoring layer

D_c = Diameter of armor material

Δp = Decimal percentage of original bed material larger than D_c

Figure 19. - Armoring definition sketch.

The sediment particle sizes required for armoring can be computed by several methods with each regarded as a check on the other. Each method will indicate a different armoring size which requires experience and judgment in selecting the most appropriate. The basic data required to make these computations necessitate: (1) samples of the streambed material through the reach involved and at a depth through the anticipated scour zone, (2) selection of a dominant discharge usually approximately a 2-year frequency peak discharge, and (3) average channel hydraulic properties for the selected dominant discharge obtained from steady flow backwater computations through the selected reach of river. The methods used to compute a nontransportable size are usually based on some form of a sediment transport equation or relationship that will form the armoring layer as described in reports of Pemberton and Lara (1982), ASCE (1975), or Yang (1973) such as those of:

	Method
1	Meyer-Peter, Muller (Sheppard, 1960) 1948
2	Competent bottom velocity (Mavis and Laushey, 1948)
3	Tractive force (Bureau of Reclamation, 1952)
4	Shields diagram (Pemberton and Lara, 1982; ASCE, 1975)
5	Yang incipient motion (Yang, 1973)

An example of a degradation computation limited by armoring using above methods is given below. The following data are known for the example computations for a channel downstream of a storage dam:

Dominant discharge = Q = 500 ft^3/s (14.2 m^3/s)
Channel width = B = 60 ft (18.3 m)
Mean channel depth = d = 4 ft (1.22 m)
Mean channel velocity = V = 3.4 ft/s (1.04 m/s)
Stream gradient = S = 0.0021
Armoring size = D = diameter in millimeter

(1) Meyer-Peter, Muller

$$D = \frac{Sd}{K\left(\frac{n_s}{D_{90}^{1/6}}\right)^{3/2}} \tag{14}$$

where

K = 0.19 inch-pound units (0.058 metric units)
n_s = 0.03 (assumed for this example)
D_{90} = 34 mm (assumed for this example)

$$D = \frac{0.0021\ (4.0)}{0.19\left(\frac{0.03}{34^{1/6}}\right)^{3/2}} = \frac{0.0048}{0.000409} = 20\text{ mm}$$

$$= \left[\frac{0.0021\ (1.22)}{0.058\ (0.00215)} = \frac{0.00256}{0.000125} = 20\text{ mm}\right]$$

(2) Competent bottom velocity

$$V_b = 0.51\ (D)^{1/2}\ \text{ft/s},\ (0.155\ (D)^{1/2}\ \text{m/s}) \qquad (15)$$

where

V_b = competent bottom velocity = 0.7 (V_m)
V_m = mean velocity ft/s (m/s)
D = diameter in millimeters
D = 3.84 V_b^2 inch-pound units
= (41.6 V_b^2) metric units
D = 3.84 $(0.7 \times 3.4)^2$ = 22 mm
= [41.6 $(0.7 \times 1.04)^2$ = 22 mm]

(3) Critical tractive force

$$\text{t.f.} = \Upsilon_w dS \qquad (16)$$

where

t.f. = tractive force, in lb/ft^2 (g/m^2)
Υ_w = unit weight (mass) of water 62.4 lb/ft^3 (1.0 t/m^3)
d = mean water depth, ft (m)
S = stream gradient

t.f. = 62.4 x 4.0 x 0.0021 = 0.524 lb/ft^2
= [10^6 g/m^3 x 1.22 x 0.0021 = 2560 g/m^2]

From figure 20, D = 31 mm

(4) Shields diagram for material >1.0 mm and Reynold's number R* >500.

$$\frac{T_c}{(\Upsilon_s - \Upsilon_w)\ D} = 0.06 \qquad (17)$$

where

T_c = critical shear stress = $\Upsilon_w dS$, lb/ft^2 (t/m^2)
Υ_s = unit weight (mass) of the particle = 165 lb/ft^3 (2.65 t/m^3)
Υ_w = unit weight (mass) of water = 62.4 lb/ft^3 (1.0 t/m^3)
d = mean water depth, ft (m)
S = slope, ft/ft (m/m)
D = Diameter of particle, ft (m)

$$D = \frac{62.4\ (4.0)\ (0.0021)}{0.06\ (165 - 62.4)} = 0.0848\ \text{ft} = 26\ \text{mm}$$

$$D = \frac{1.0\ (1.22)\ (0.0021)}{0.06\ (1.65)} = 0.0259\ \text{m} = 26\ \text{mm}$$

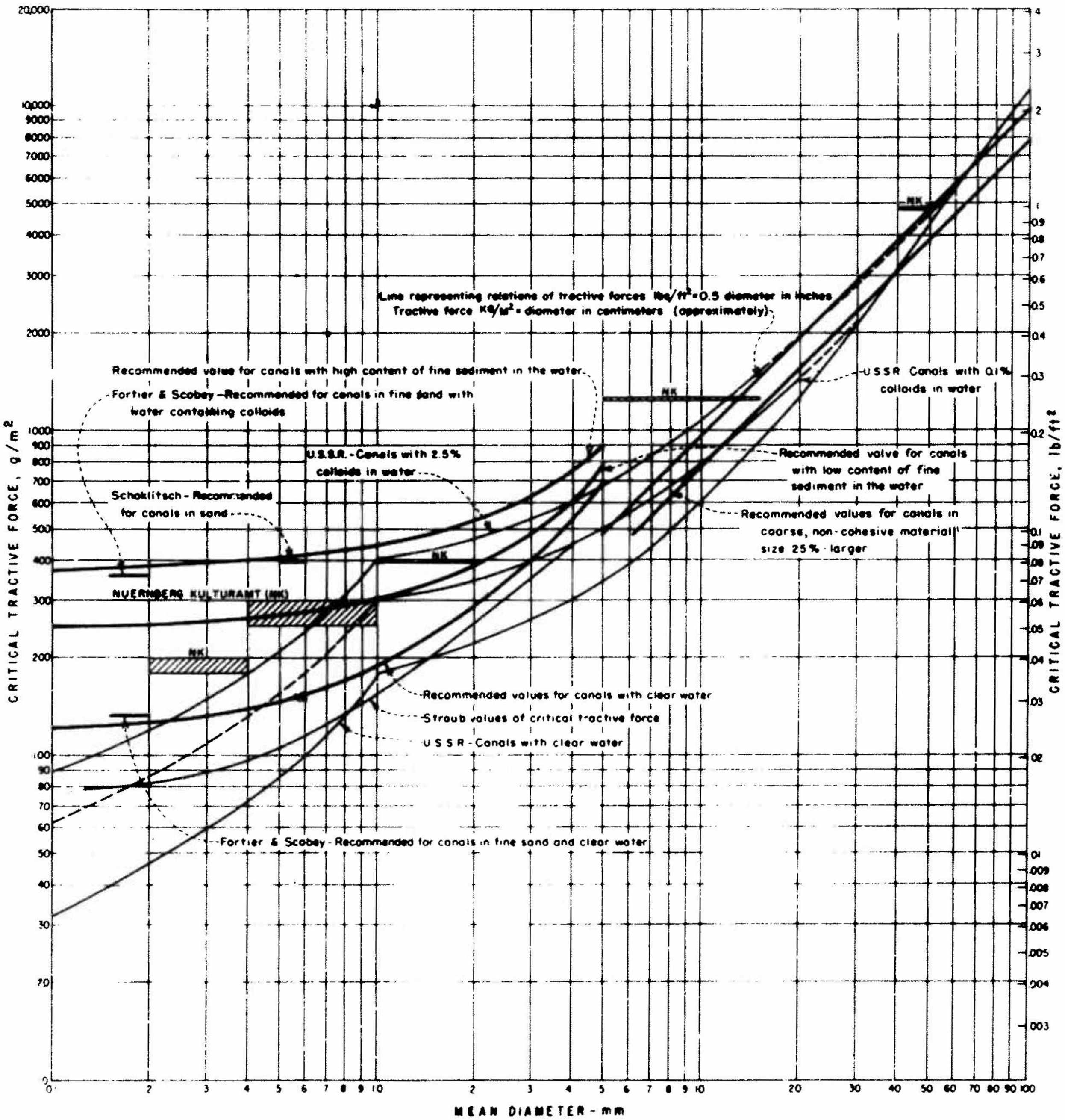


Figure 20. - Tractive force versus transportable sediment size.

(5) Yang incipient motion criteria for shear velocity Reynold's number $R_{\star}>70$:

$$\frac{V_{cr}}{w} = 2.05 \tag{18}$$

where

V_{cr} = critical average water velocity at incipient motion, ft/s (m/s)
w = terminal velocity, ft/s (m/s),

with Rubey (1933) settling velocity for materials larger than about 2 mm in diameter, the fall velocity can be approximated by:

$$\begin{aligned} w &= 6.01\ D^{1/2} \text{ inch-pound units} \\ (w &= 3.32\ D^{1/2}) \text{ metric units} \end{aligned} \tag{19}$$

Equations 18 and 19 can be combined to give:

$$\begin{aligned} D &= 0.00659\ V_{cr}^{2} \text{ inch-pound units} \\ (D &= 0.216\ V_{cr}^{2}) \text{ metric units} \end{aligned} \tag{20}$$

in the example problem

$$\begin{aligned} D &= 0.00659\ (3.4)^2 = 0.762 \text{ ft} = 23 \text{ mm} \\ [D &= 0.216\ (1.04)^2 = 0.0233 \text{ m}] = 23 \text{ mm} \end{aligned}$$

Mean of the above five methods for computing armoring size is 24 mm, which was adopted as a representative armoring size. By use of equation 13, an assumed three layers of nontransportable material to form an armor, and assumed 17 percent of bed material >24 mm (from size analysis of streambed material), the depth of degradation is:

$$y_a = 3\ D = 3(24) = 72 \text{ mm} = 0.236 \text{ ft } (0.072 \text{ m})$$

$$y_d = 0.236 \left(\frac{1}{0.17} - 1\right) = 1.15 \text{ ft}$$

$$= (0.072 \left(\frac{1}{0.17} - 1\right) = 0.35 \text{ m})$$

c. Stable slope method. - The method of computing a stable slope to define degradation below a dam is used when there is not enough coarse material to develop an armoring layer. The method is used when the primary purpose is to compute a depth of scour immediately below the dam for design of the dam and downstream protection against vertical scour of the streambed. It is also used in early planning stages with a limited amount of field data and when costs for the more detailed study are prohibitive. The more detailed electronic computer solutions (Corps of Engineers, 1977) or their predecessor, the desk calculator method by Lane (1948), are used when data are available to verify the mathematic model, channel hydraulics can be easily synthesized, and degradation results influence the channel morphology for several miles (kilometers) below the dam.

The stable slope method is illustrated by the sketch in figure 21. The stable slope is defined as the slope of the stream at which the bed material will no longer be transported. As shown by figure 21, the method is also identified as the three-slope method because of the variation expected in slope between the stable slope and the existing slope further downstream. The computations of stable slope can be made by application of several methods such as: (1) Schoklitsch (Shulits, 1935) bedload equation for conditions of zero bedload transport, (2) Meyer-Peter, Muller (1948) and by Sheppard (1960) bedload equation for beginning transport, (3) Shields diagram (Pemberton and Lara, 1982; ASCE, 1975) for no motion, and (4) Lane (1952) relationship for critical tractive force assuming clear water flow in canals. The discharge to be used in any of the above methods is the dominant discharge and is usually determined by the channel bankfull flow or 2-year flood peak discharge. With regulation of the streamflow by an upstream dam, the problem becomes more complex because detailed data on future releases are usually not available. If the releases from the reservoir are fairly uniform, and flood discharges are a relatively rare occurrence, the average daily discharge may be used as the dominant discharge. However, if the releases are subject to considerable fluctuation due to floods, the peak discharge which is equaled or exceeded on the average of once every 2 years would be considered the dominant discharge.

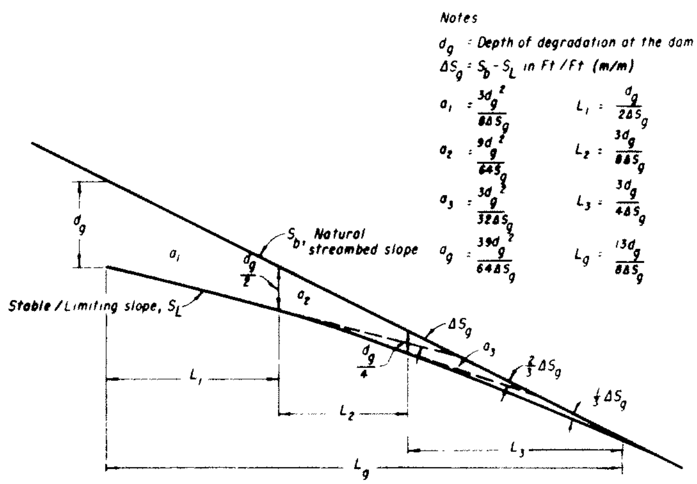


Figure 21. - Degraded channel by the three-slope method.

The next step in degradation computations by the stable slope method is determination of the average channel hydraulic properties for the dominant discharge. These data can usually be obtained from the tailwater study that has been prepared for the dam. The properties of all the tailwater cross sections when carrying the dominant discharge are averaged to arrive at a generalized cross section which will be representative of the degradation reach. The water surface slope may be assumed equal to the hydraulic gradient.

The volume of material to be removed by the stable slope method can be determined in several ways. From figure 21 it may be expressed by:

$$V_g = a_g B_d \tag{21}$$

where

V_g = volume of material to be degraded in ft^3 (m^3)
a_g = longitudinal area of degradation in ft^2 (m^2)
B_d = degraded channel width in feet (m)

If there are no downstream controls or no limit to the length (L_g) for degradation, the two ways to compute the volume are: (1) assume the river will pick up a load of coarse sediment (>0.062 mm) equal to that portion of the historic sediment load >0.062 mm, or (2) compute the outflow from the degraded reach by a sediment-rating curve, flow-duration curve method. In the second case, the sediment-rating curve would be defined by use of one or more of the bedload equations and the flow-duration curve of anticipated reservoir releases.

By rearranging equation 21 the longitudinal area may be found:

$$a_g = \frac{V_g}{B_d} \tag{22}$$

Once a value has been found for a_g, the depth of degradation may be computed using the following equation:

$$d_g = \left(\frac{64\ a_g\ \Delta S_g}{39}\right)^{1/2} \tag{23}$$

where

ΔS_g = the difference between the existing slope and the stable slope, and the length of the degraded reach can be computed by:

$$L_g = \frac{13\ d_g}{8\,\Delta S_g} \tag{24}$$

If it is anticipated that lateral degradation will be a significant factor, additional study will be necessary to determine the degraded channel width. Because part of the material will be coming from the streambanks, the extent of vertical degradation will ordinarily not be as great. Lateral movement should always be suspect when the banks are composed of the same material as the bed and there is not a great deal of vegetation to hold them. Procedures described by Pemberton and Lara (1982), Bureau of Reclamation (1951) and (1952) are recommended as guides when these conditions prevail.

If a permanent control exists at some point within the degradation reach, equation 24 may be used to solve for the depth of degradation directly.

The three-slope or stable slope method for computing the depth of degradation at the dam and the degraded channel profile are based on satisfying the following assumptions:

1. The degradation reach is sufficiently uniform to permit the use of average cross sections and slope throughout the reach.

2. The bed and bank material throughout the reach is similar enough that an average composition can be used and that there are no existing nonerodible barriers in the bed or banks to prevent the stream from attaining the average section at the stable slope.

3. The degradation will be such that the vertical component will predominate and horizontal movement will be limited to bank sloughing resulting from vertical degradation.

The Meyer-Peter, Muller equation for beginning transport has been selected in an example problem for degradation computations by the stable slope method. Computations by use of the other more commonly used methods for computing a stable slope are described by Pemberton and Lara (1982). The following data are known about a river channel below a diversion dam:

Dominant discharge = Q = 780 ft^3/s (22.1 m^3/s)
Channel width = B = 350 ft (107 m)
Mean channel depth = d = 1.05 ft (0.32 m)
Existing stream gradient = S = 0.0014
Bed material $D_m = D_{50} = 0.3$ mm
$D_{90} = 0.96$ mm
Manning's "n" for bed of stream, $n_s = 0.027$

Preliminary studies show that 2160 acre-feet (2.66×10^6 m^3) of sand would deposit behind the diversion dam during the 100-year economic life of the structure. Investigations support an equal volume of sand could be eroded from the downstream channel.

The stable slope computation by the Meyer-Peter, Muller equation for beginning transport is:

$$S_L = \frac{K \frac{Q}{Q_B} D \left(\frac{n_s}{D_{90}^{1/6}} \right)^{3/2}}{d} \qquad (25)$$

where

S_L = limiting slope
K = 0.19 inch-pound units (0.058 metric units)

$\frac{Q}{Q_B}$ = Ratio of total flow in ft^3/s (m^3/s) to flow over bed of stream in ft^3/s (m^3/s). Usually defined at dominant discharge where $\frac{Q}{Q_B}$ = 1 for wide channels.

$$S_L = \frac{0.19\ (0.3)\left(\frac{0.027}{0.96^{1/6}}\right)^{3/2}}{1.05} = 0.00024$$

$$= \left(\frac{0.058\ (0.3)\ (0.00448)}{0.32}\right) = 0.00024$$

The difference between the existing and degraded slope, ΔS_g, is 0.00116. The longitudinal degradation area by equation 22 is:

$$a_g = \frac{43\ 560\ (2160)}{350} = 269\ 000\ ft^2$$

$$= \frac{2.66 \times 10^6}{107} = 24\ 900\ m^2$$

The depth of degradation at the dam by equation 23 is:

$$d_g = \left(\frac{64\ (269\ 000)\ (0.00116)}{39}\right)^{1/2} = 22.6\ ft$$

$$\left[= \left(\frac{64\ (24\ 900)\ (0.00116)}{39}\right)^{1/2} = 6.88\ m\right]$$

and the length by equation 24 of the degradation reach is:

$$L_g = \frac{13\ (22.6)}{8\ (0.00116)} = 31\ 700\ ft$$

$$= \left(\frac{13\ (6.88)}{8\ (0.00116)} = 9640\ m\right)$$

Conclusions. - The procedures described in these guidelines for presenting sediment inflow will depend on sediment yields and available sediment data. In those cases requiring collection of suspended sediment samples, a 5-year program of either daily or less frequent sampling on an intermittent basis can be used to define a sediment-rating curve. This curve, combined with a flow-duration curve, represents the best method for determining the 100-year sediment inflow. Some estimate is necessary for the bedload which can be made by correction factor or one or more of the equations cited in this report.

Reservoir sediment distribution techniques are described for allocation and location of deposited sediments. The methods given for estimating density of deposited sediments and trap efficiencies will vary from one reservoir to another. Every situation is unique, and the methods selected for estimating total sediment inflow, sediments deposited, and distribution will vary.

Prediction of the degradation in the river channel below a dam may involve the application of a sophisticated mathematical model. These models are still undergoing development and change besides being costly and subject to limitations because of basic data. Certain situations may still warrant the use of the less detailed models like the limiting slope method. If armoring is anticipated, the armoring analysis is judged satisfactory.

References. -

Ackers, P., and W. R. White, "Sediment Transport: New Approach and Analysis," Journal, Hydraulics Division, Proceedings, American Society of Civil Engineers, , vol. 99, No. HY 11, November 1973.

ASCE, "Sedimentation Engineering," American Society of Civil Engineering Manual No. 54, V. A. Vanoni, editor, 1975.

Blanton, J. O. III, "Procedures for Monitoring Reservoir Sedimentation," Bureau of Reclamation, October 1982.

Borland, W. M., and C. R. Miller, "Distribution of Sediment in Large Reservoirs," Transactions, American Society of Civil Engineers, vol. 125, 1960.

Brune, Gunnar M., "Trap Efficiency of Reservoirs," Transactions of American Geophysical Union, vol. 34, No. 3, June 1953.

Bureau of Reclamation, "Computation of 'Z's' for Use in the Modified Einstein Procedure," Denver, Colorado, June 1966.

Bureau of Reclamation, "Progress Report on Results of Studies on Design of Stable Channels," Hydraulic Laboratory Report Hyd 352, Denver, Colorado, June 1952.

Bureau of Reclamation, "Stable Channel Profiles," Hydraulic Laboratory Report No. Hyd. 325, Denver, Colorado, September 1951.

Bureau of Reclamation, "Step Method for Computing Total Sediment Load by the Modified Einstein Procedure," Denver, Colorado, July 1955.

Chow, Ven Te, editor-in-chief, Handbook of Applied Hydrology, McGraw-Hill, New York, N.Y., 1964.

Churchill, M. A., Discussion of "Analysis and Use of Reservoir Sedimentation Data," by L. C. Gottschalk, pp. 139-140, Proceedings, Federal Interagency Sedimentation Conference, Denver, Colorado, January 1948.

Colby, B. R., and C. H. Hembree, "Computations of Total Sediment Discharge, Niobrara River near Cody, Nebraska," U.S. Geological Survey Water Supply Paper No. 1357, 1955.

Corps of Engineers, "Scour and Deposition on Rivers and Reservoirs - HEC-6," User's Manual for a Generalized Computer Program, Exhibit 3, 1977.

Einstein, Hans Albert, "The Bed-load Function for Sediment Transportation in Open Channel Flows," U.S. Soil Conservation Service Technical Bulletin No. 1026, September 1950.

Emmett, W. W., "A Field Calibration of the Sediment-Trapping Characteristics of the Helley-Smith Bed-load Sampler," U.S. Geological Survey Professional Paper No. 1139, 1980.

Engelund, F., and E. Hansen, "A Monograph on Sediment Transport in Alluvial Streams," Teknisk Forlag, Denmark, 1967.

Herbertson, John G., "A Critical Review of Conventional Bed-load Formulae," Journal of Hydrology, vol. VIII, No. 1, May 1969.

Hudspeth, C. L., and J. W. Trietsch, "User's Manual - SWE1, Computer Program for Sediment Distribution," unpublished report by Bureau of Reclamation, SW Region, Amarillo, Texas, January 1978.

Lane, E. W., "An Estimate of the Magnitude of the Degradation Which Will Result in the Middle Rio Grande Channel from the Construction of the Proposed Sediment Storage Basins and Contraction Works," Bureau of Reclamation, Hydraulic Laboratory Report Hyd 290, 1948.

Lane, E. W., "The Importance of Fluvial Morphology in Hydraulic Engineering," Journal, Hydraulic Division, Proceedings, American Society of Civil engineers, vol. 81, July 1955.

Lane, E. W., and V. A. Koelzer, "Density of Sediments Deposited in Reservoirs," In: A Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams, Report No. 9, Interagency Committee on Water Resources, 1943.

Lara, J. M., "Revision of the Procedure to Compute Sediment Distribution in Large Reservoirs," Bureau of Reclamation, May 1962.

Lara, J. M., and E. L. Pemberton, "Initial Unit Weight of Deposited Sediments," Proceedings, Federal Interagency Sedimentation Conference, 1963, Miscellaneous Publication No. 970, pp. 818-845, U.S. Agriculture Research Service, June 1965.

Lara, J. M., and H. I. Sanders, "The 1963-64 Lake Mead Survey," Bureau of Reclamation, 1970.

Mavis, F. T., and L. M. Laushey, "A Reappraisal of the Beginning of Bed-Movement-Competent Velocity," International Association for Hydraulic Research, Second meeting, Stockholm, June 1948.

Meyer-Peter, E., and R. Muller, "Formulas for Bed-load Transport," Meeting of the International Association for Hydraulic Structure Research, Second meeting, Stockhom, June 1948.

Miller, C. R., "Analysis of Flow-Duration, Sediment-Rating Curve Method of Computing Sediment Yield," Bureau of Reclamation, Denver, Colorado, 1951.

Miller, Carl R., "Determination of the Unit Weight of Sediment for Use in Sediment Volume Computations," Bureau of Reclamation, Denver, Colorado, 1953.

Murthy, B. N., "Life of Reservoir," Technical Report No. 19, Central Board of Irrigation and Power, New Delhi, reprinted September 1980.

Pacific Southwest Interagency Committee, "Factors Affecting Sediment Yield and Measures for the Reduction of Erosion and Sediment Yield," October 1968.

Pemberton, E. L., "Channel Changes in the Colorado River Below Glen Canyon Dam," Proceedings, Third Federal Interagency Sedimentation Conference, 1976.

Pemberton, E. L., "Einstein's Bed-load Function Applied to Channel Design and Degradation," Chapter 16, In: Sedimentation, Symposium to honor Prof. H. A. Einstein, H. W. Shen, editor, 1972.

Pemberton, E. L. and J. M. Lara, "Guide for Computing Degradation and Local Scour," Bureau of Reclamation, Denver, Colorado, October 1982.

Rubey, W. W., "Settling Velocities of Gravel, Sand, and Silt Particles," American Journal, Science, 1933.

Sheppard, John R., "Investigation of Meyer-Peter, Muller Bed-load Formulas," Bureau of Reclamation, June 1960.

Shulits, Samuel, "The Schoklitsch Bed-load Formula," Engineering, vol. 139, 1935.

U.S. Department of Agriculture, "Sediment Deposition in U.S. Reservoirs, Summary of Data Reported Through 1975," Miscellaneous Publication No. 1362, Agriculture Research Service, February 1978.

U.S. Interagency Sedimentation Project, "A Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams," Reports No. 1 through 14 and A through W, Subcommittee on Sedimentation, 1940 to 1981.

U.S. Government Handbook, "Chapter 3 - Sediment, National Handbook of Recommended Methods for Water-Data Acqusisiton," 1978.

Wischmeier, W. H., and D. D. Smith, "Predicting Rainfall Erosion Losses from Cropland East of the Rocky Mountains," Agric. Handbook 282, U.S. agriculture Research Service, 1965.

Yang, C. T., "Incipient Motion and Sediment Transport," ASCE, vol. 99, Hy No. 10, October 1973.

GPO 838-290

Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.

A free pamphlet is available from the Bureau entitled, "Publications for Sale". It describes some of the technical publications currently available, their cost, and how to order them. The pamphlet can be obtained upon request from the Bureau of Reclamation, Attn D-922, P O Box 25007, Denver Federal Center, Denver CO 80225-0007.