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FIFTH SESSION - LABORATORY INVESTIGATIONS

Albert S. Fry,* Presiding

DEVELOPMENT OF THE MECHANICS OF SEDIMENT TRANSPORTATION

by Vito A. Vanoni **

Introduction

The purpose of this paper is to outline briefly the history and development of the mechanics of sediment transportation and to indicate the information in this field that is available to the engineer and sedimentation specialist. A brief chronological history of the development will be given with some discussion of the more significant results. No attempt will be made to give a complete bibliography on this extensive subject since this has been done in an excellent fashion by a number of writers to which reference will be made. Finally, lines of research will be suggested which will furnish the information that must be obtained to help solve the many sediment problems now facing the people of our country.

The History of Sediment Transportation

Duboys and Gilbert. The science of sediment transportation was started by DuBoys 1/, who, in 1879, published the formula which now carries his name. He introduced the idea of the tractive force, or the shear force, at the bed of the stream. He also introduced the idea of the critical tractive force which most workers in this field define as the tractive force at which general movement takes place in the bed of the stream. The DuBoys formula (derivation given by Rouse 2/ is

$$G = \psi \tau_o \quad (\tau_o - \tau_c) \qquad (1)$$

where G is the rate of sediment movement, in pounds per second per foot of width ψ is a coefficient, τ_o is the tractive force, or shear, at the bed and τ_c is the critical value of this force in pounds per square foot. The average shear at the bed is calculated by summing up the forces on a section of the stream one unit in length, as shown diagrammatically in Fig. 1. Assuming uniform flow, the pressure forces, F, at the two ends of the prism are the same and the only forces entering the equation are the weight of the water and the friction force along the bed. The average value of the friction force is

$$\bar{\tau}_o = \frac{\gamma AS}{p} = \gamma RS \qquad (2)$$

where γ is the specific weight of water in lbs/cu.ft.

A is the cross sectional area of the stream in square feet

p is the wetted perimeter in feet

S is the slope in feet per ft, and

R is the hydraulic radius in feet

* Chief, Hydraulic Data Division, Tennessee Valley Authority, Knoxville, Tenn.

** Project Supervisor, Cooperative Laboratory, Soil Conservation Service, and California Institute of Technology, Pasadena, Calif.

1/ 2/ See "References" at close of comments.

The work of DuBoys has been followed by practically all workers on the bed-load problem, and reference can be found to his work in most publications in this field.

The next work of major importance in this field was that by G. K. Gilbert 3/, who did his work in about 1909 at the University of California and published it in 1914. Gilbert was one of the noted geologists of his time, and judging from his work on sediment transportation, he was an extremely able observer and research man. He pioneered in making sediment transportation experiments in flumes and many of his techniques are still being used. Although he did not develop a sediment transportation formula, he made some very comprehensive experiments, the data from which have been used over and over again. (Johnson 4/ has compiled all available bed-load data). Gilbert described very clearly the modes of movement. He observed that for incipient motion, the grains of sand are moved by rolling and sliding along the bed. As the velocity increases, he noted that the grains tend to take short jumps in curved orbits. He described this mode of transportation as saltation. As the flow was increased further, he noted that material was carried in suspension. He described in detail the sand waves similar to aeolian dunes which formed at the bed soon after incipient motion. As these dunes progress downstream the material is carried up the flat, upstream slope and deposited on the steep downstream slope. As the flow increases, the dunes are flattened and the sediment moves over the entire bed without any waves. When the velocity is increased further, another type of bed undulations appear, which he termed "anti-dunes." These dunes or sand waves move upstream and are periodic in nature, tending to form, build up to a maximum, and then disappear. They are accompanied by waves on the water surface similar in shape to the bed wave. Gilbert also introduced the term, "competence," which is applied to the characteristics of the stream for the conditions at which sediment commences to move. For instance, for a given slope and channel, the competent flow is that at which the bed material will begin to move.

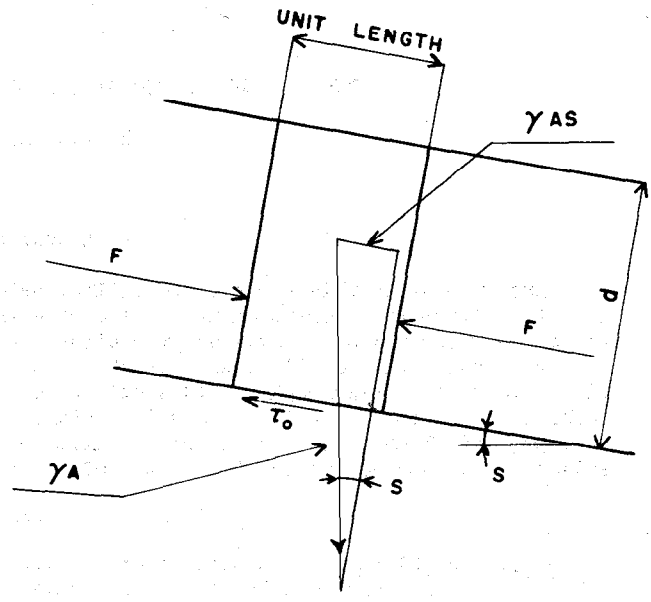


Fig. 1. Tractive Force on Channel Bottom.

Development Following World War I

Following World War I, the problem of transportation of material by streams was attacked vigorously both in Europe and in the United States. During this period also the movable bed hydraulic model was developed and much of the research on sediment transportation was in connection with developing techniques of representing alluvial streams to a small scale. One of the principal factors in the development of sediment transportation and hydraulic engineering in general in the United States was the establishment of travel fellowships by the late John R. Freeman. Through this means many promising young men studied in Europe where the science of hydraulics and the use of the model was developed much more than it was in this country. The inspiration obtained in this manner was responsible largely for the sediment studies in this country.

The result of this activity was the appearance of quite a few bed-load formulas of an empirical or semi-empirical nature, based on sediment transportation experiments in flumes. A number of the better known of these formulas are shown in Table 1. An inspection of these formulas will show that most of them are similar to the DuBoys formula and that they involve either a critical tractive force, or a critical flow. A study of the results of the various investigations will show that it is difficult to get a precise value of the critical tractive force or flow. The critical values are defined as the values existing when general movement first occurs. The term, "general movement," is difficult to define precisely and different observers are apt to have different interpretations of the term. The result is that the data of different workers are not always comparable. The fact that so many formulas appeared is very confusing. This confusion was clarified to a great extent by Johnson 5/, who compared a number of these formulas by plotting the same data according to a number of different formulas. He did this for data from

3/ 4/ 5/ See "References" at close of comments.

Table 1. Formulas for Rate of Bed-Load Transportation

Worker	Formula	Sediment	Eqn. No.
U. S. Waterways Experiment Station (4)	$G = \frac{K}{n}(\tau_o - \tau_o)^m$	Sand mixtures	(3)
Chang (5)	$G = Kn\,\tau_o(\tau_o - \tau_o)$	Uniform sand	(4)
O'Brien (6)	$G = A\left(\frac{V}{R^{1/3}}\right)^m$	Sand mixtures	(5)
MacDougall (7)	$G = AS^m(Sq - Sq_c)$	Sand mixtures	(6)
Schoklitsch (8)	$G = \frac{A}{\sqrt{D}}\,S^{3/2}(q - q_c)$ $q_c = \frac{BD}{S^{4/3}}$	Uniform sand	(7)
Meyer-Peter (11)	$G = (BS\,q_1^{2/3} - B_1D)^{3/2}$	Uniform sand Large size	(8)

LEGEND

G = Rate of sediment movement in lbs./sec/ft of width

n = Mannings roughness coefficient

τ_o = tractive force at bed in lbs/sq.ft.

τ_c = critical tractive force at bed in lbs/sq.ft.

D = diameter of sand in mm.

S = slope of the energy gradient in feet per foot

q = rate of flow per unit width in cu. ft. per sec. per ft. of width

q_o = critical value of flow per unit width or flow when general movement of the bed material first starts - cu. ft. per sec. per ft. of width

q_1 = rate of flow per unit width in lbs per sec per ft of width

K, A, and B = empirical constants

m = empirical exponent

the U. S. Waterways Experiment Station 6/ for Formulas 1, 3, 4, 5, 6 and 8. Fig. 2 shows his graphs for Formulas 1, 3 and 4. It is very difficult from inspection to determine which of the formulas gives the best fit. By means of a statistical analysis of the various graphs, Johnson concluded that the goodness of fit for the three graphs shown in Fig. 2, as well as for graphs of equations 5, 6, and 8, was about the same. From this he concluded that the choice of equations could be made on the basis of the convenience in measuring the variables appearing in the formula.

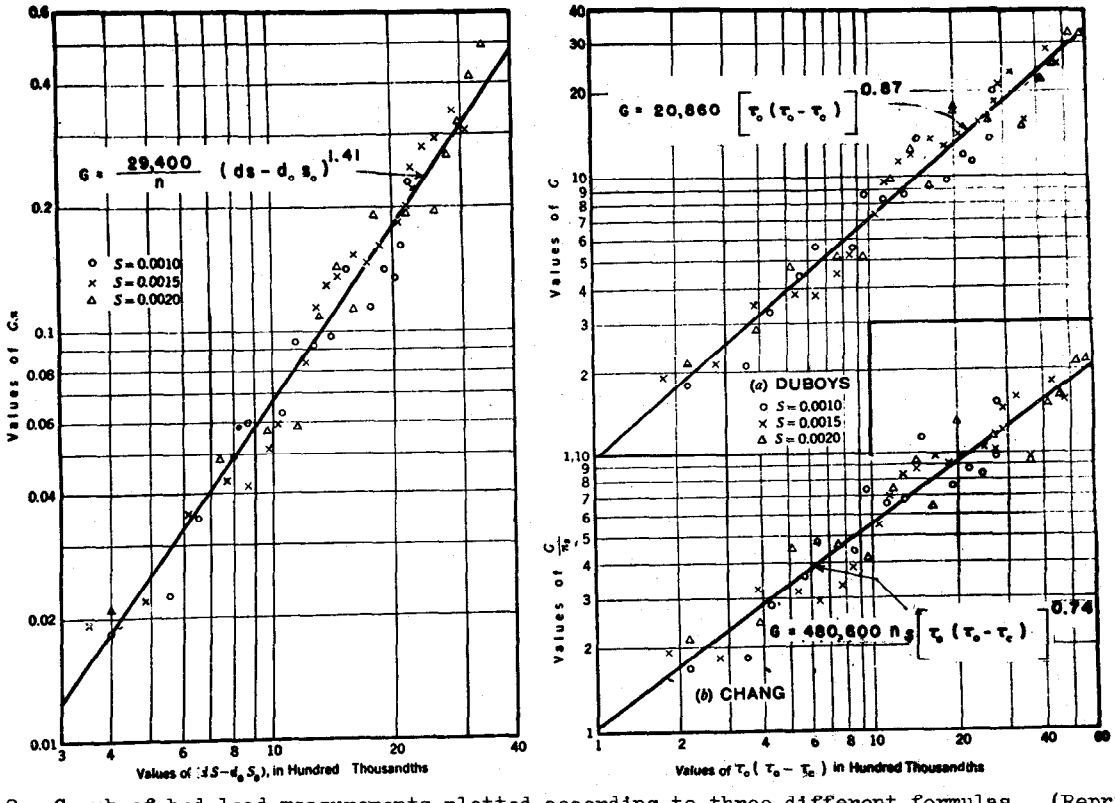


Fig. 2. Graph of bed-load measurements plotted according to three different formulas. (Reprinted from "Laboratory Investigations of Flume Traction and Transportation" (See "References," item 7, at close of comments)"

6/ See "References" at close of comments.

An inspection of the bed-load formulas in Table 1 will show that there are a number of constants in the equations which must be determined empirically before the equation can be applied. To begin with, the critical values of either the shear or the discharge must be determined. The difficulty in the definition of critical conditions makes it very difficult to determine these values and leads to inconsistencies between various workers. The same difficulties appear to apply to the determination of the other constants. The general impression that is obtained in looking at the formulas is that they are too simple to represent such a complicated system as a sediment-carrying stream. It is likely that some of the constants are actually functions.

As mentioned previously, much of the work on sediment transportation was done to develop techniques for movable bed model studies. The comprehensive work of Kramer 12/ and of the U. S. Waterways Experiment Station was done for this purpose. The objective was to find a sand mixture that could be used in a river model to simulate movement for all rates of flow occurring in the prototype. Kramer was one of the first workers to introduce the size frequency distribution or size grading of the sediment as a variable. He expressed the size grading empirically by the ratio of the areas above and below the 50 per cent line on the standard mechanical analysis graph where grain diameter is plotted against the percentage of the sample that is finer than the given diameter. He used this ratio in his expression for critical tractive force.

Newer Theories

Starting in about 1935, new approaches to the problem were developed by applying developments in basic fluid mechanics that appeared a few years before this time as a result of rapid developments in aeronautics. Shields13/ showed that the tractive force for initial movement could be expressed as a function of the dimensionless parameter $\sqrt{\frac{\tau_o}{\rho}}\,D/\nu$ where D is the diameter of the sediment grains in feet, ρ is the mass density of the fluid in slugs per cubic and ν is the kinematic viscosity of the fluid in square ft/sec. This parameter is proportional to D/δ where δ is the thickness of the thin layer of laminar flow that occurs at the boundaries of a channel. The value of δ is given by the following formula derived by von Karman in 1930,

$$\delta = 11.6\,\frac{\nu}{\sqrt{\frac{\tau_o}{\rho}}}$$

Fig. 3 is Shields' graph of dimensionless critical tractive force plotted against the parameter $\sqrt{\frac{\tau_o}{\rho}}\,D/\nu$ which is proportional to $\frac{D}{\delta}$. As this graph shows, he was also able to correlate the dune formation at the bed with the parameter - an extremely significant result, especially in view of the wide range of the variables which he covered. He varied the density of his sediments from 1.06 to 4.2 grams per cc and the diameter of his sediments from 0.36 to 3.4

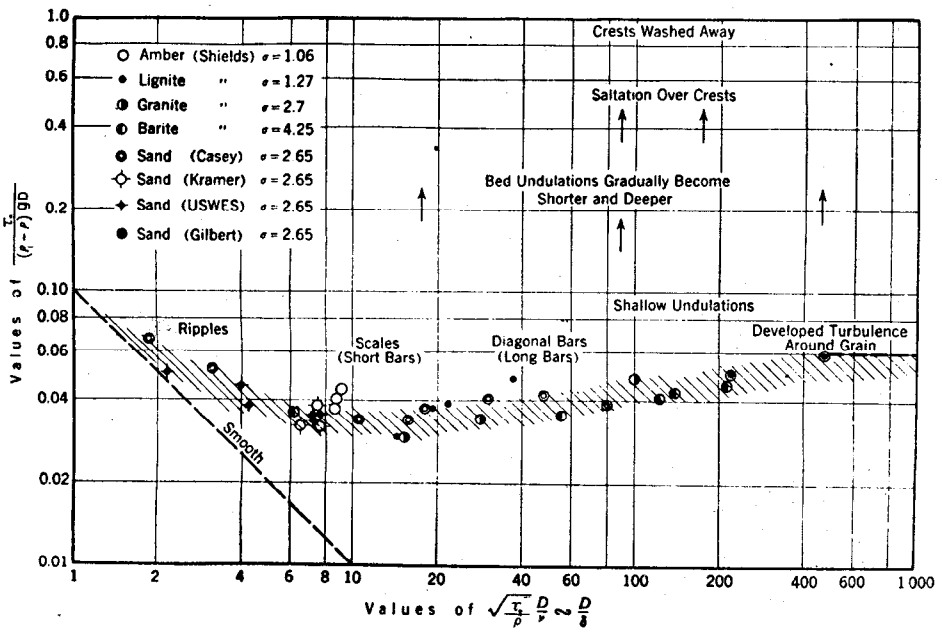


Fig. 3. Tractive force plotted against Reynolds number of sand grain according to Shields. (Reprinted from "Laboratory Investigations of Flume Traction and Transportation") (See "References," item 7, at close of comments.

12/ 13/ See "References" at close of comments

mm. He classifies sand waves at the bed into ripples, short bars and long bars in decreasing order of height to length ratio, and shows in Fig. 3 that they can be classified according to D/δ. Shields states that any bed-load formula must contain parameters which will describe the bed configuration much as Fig. 3 does. He also develops an empirical bed-load formula which applies only to sediments which do not form ripples or high waves.

In 1941 H. A. Einstein 14/ published his ϕ - ψ formula for bed-load. It represented a radical departure from all of the previous bed-load formulas in that it was not based on the conception of a critical tractive force or critical condition of any kind. It is based on two main ideas. The first is that the sediment moves in steps or jumps and that the length of the step, L, is given by

$$L = \lambda D$$

where D is the diameter of the sediment grain and

λ is a coefficient

The second is that the number of particles that will move out of an area of unit width and length, L, in a second depends on the probability that the lift force will exceed the weight of the particle in water. By developing this idea with the use of dimensional analysis, he arrives at the formula.

$$A\phi = f(B\psi) \tag{9}$$

$$\text{where} \quad \phi = \frac{1}{F} \frac{G}{p_1 g} \sqrt{\frac{p}{p_1 - p}} \frac{1}{g^{0.5} D^{1.5}}, \tag{10}$$

$$\psi = \frac{p_1 - p}{p} \frac{D}{RS}, \tag{11}$$

$$F = \sqrt{\frac{2}{3} + \frac{36\mu^2}{gD^3 p(p_1 - p)}} - \sqrt{\frac{36\mu^2}{gD^3 p(p_1 - p)}} \tag{12}$$

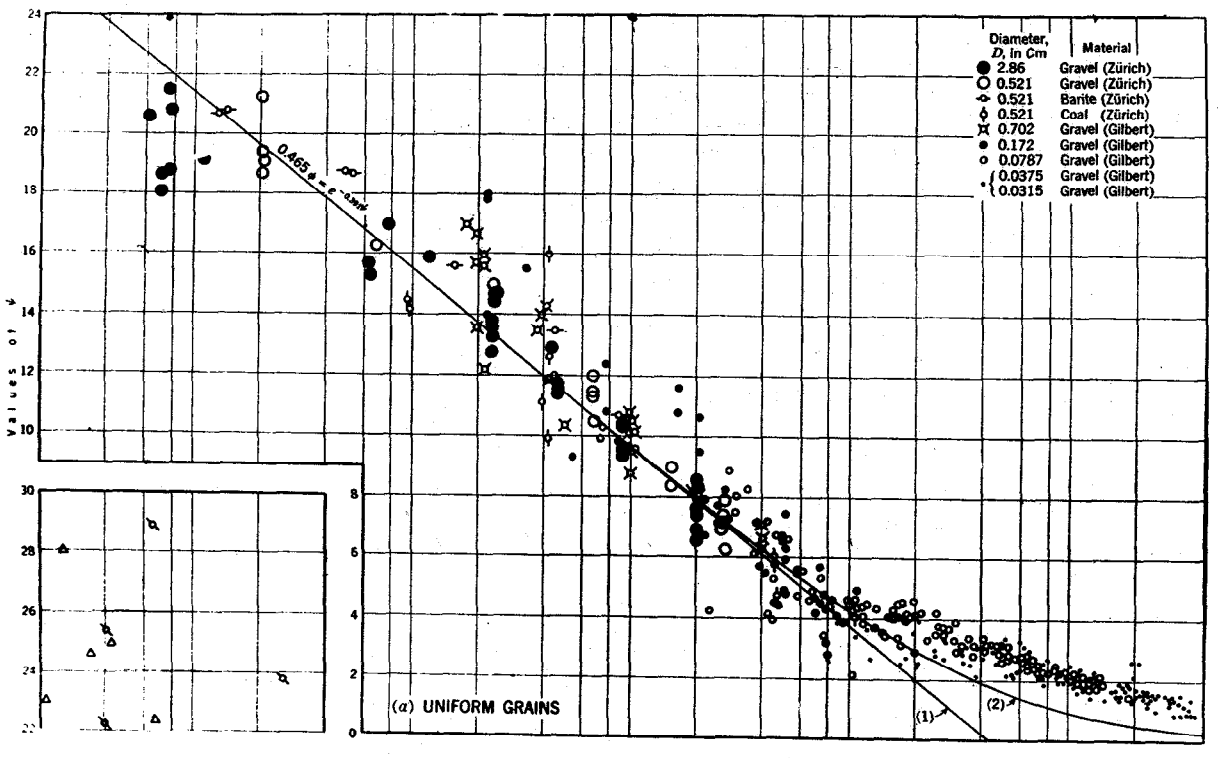
p_1 is the mass density of the sediment in slugs per cu.ft., g is the acceleration of gravity in ft/sec per second, μ is the coefficient of viscosity in pound seconds per sq.ft., and A and B are constants which are to be determined experimentally along with the function f. Fig. 4 is a graph showing data from bed-load experiments by various investigators plotted according to Einstein's formula. Fig. 4a, which is for uniform size material, follows a curve very well. For values of ϕ up to 0.4, the formula can be expressed as

$$0.465\,\phi = e^{-0.391\,\psi} \tag{13}$$

The fact that the results from various workers follow a curve is remarkable since the data cover experiments with material varying from 28.6 to 0.35 mm. in diameter, a variation of almost one hundredfold and for flows varying in depth from .06 to 3.6 feet. Fig. 4b is a graph of the data obtained by the U. S. Waterways Experiment Station on sand mixtures. The representative diameter of the sand mixtures was taken as that diameter for which 40 per cent of the material is finer or the sieve size through which 40 per cent of the material will pass. The data seem to follow a functional trend except that there is a range between curve (S) and (1) where the data do not fit a curve. Einstein attributed this to sorting in the experiments. However, the workers at the U. S. Waterways Experiment Station did not believe that this could be possible since they were extremely careful to avoid sorting. The result is that the discrepancy is not explained satisfactorily.

Einstein 14/ developed a method for determining the true shear on the bed by correcting for the friction on the walls. His formula and the Meyer-Peter formula, equation (8), are based on corrected values of R and Q, to compensate for the wall shear. The need for this was recognized by earlier workers and attempts to correct for it were made. Shoklitsch 10/ actually proposed a formula which is similar to Einstein's formula. Experimenters attempted to correct for the friction by using smooth walls so that the friction could be neglected, by roughening the walls with the same sand as in the bed and some collected the sediment only from the center portions of the flume, where the side wall effect could be neglected. Shields applied a wall shear correction in calculating values of τ_c

10/ 14/ See "References" at close of comments.



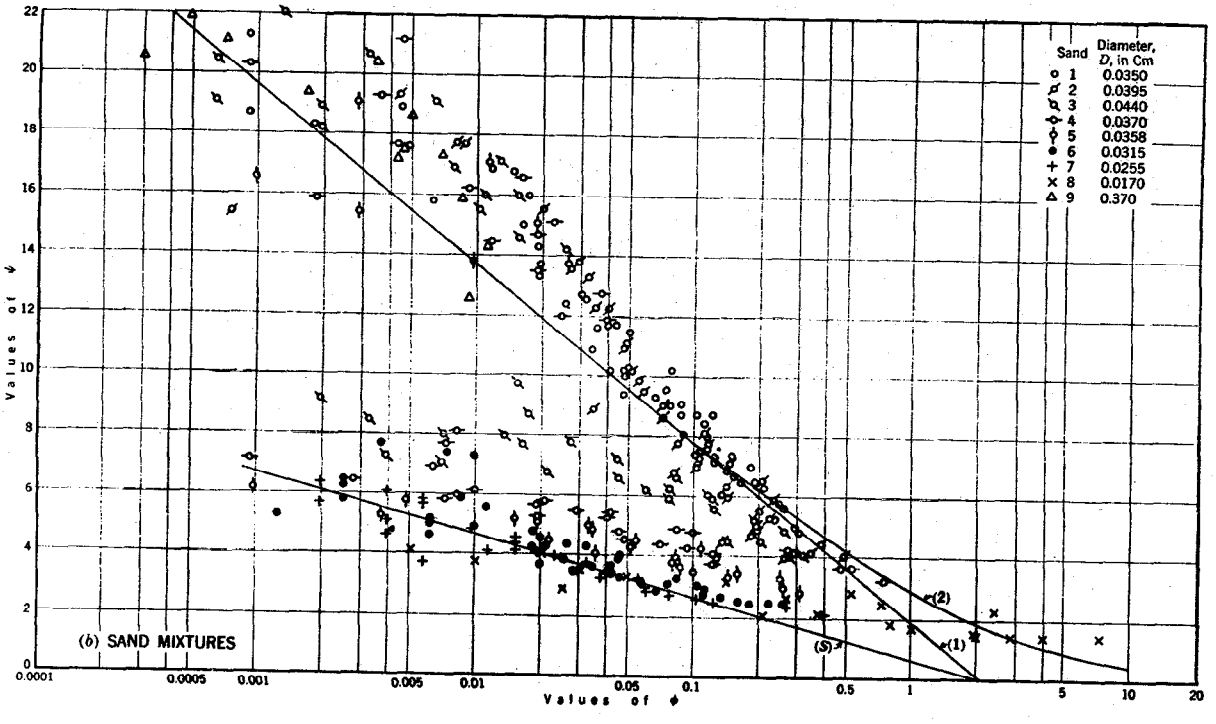


Fig. 4. Graph of Einstein ϕ - ψ bed-load formula. (a) for uniform sand, (b) for sand mixtures (Reprinted from "Formulas for the Transportation of Bed-load") (See "References," item 14, at close of comments)

Einstein 15/ carried his work farther by actually making sediment load measurements in two small streams. Mountain Creek, which is in South Carolina, is about 14 feet wide, has a slope of from .0014 to .0018 and bed material about 0.68 mm. in diameter. Measurements were made with flow depths up to about 1.4 feet. Goose Creek, in Mississippi, is about 13 feet wide, with a slope ranging from .0025 to .0031 and with material 0.25 mm. in diameter. Measurements were made in it with flow depths up to 0.6 foot. Fig. 5 shows a ϕ - ψ graph of the measurements on these streams. Curves 1 and 2 appearing in Fig. 4 have also been put on Fig. 5 for comparison. It can be seen that the field and laboratory measurements follow the same curve.

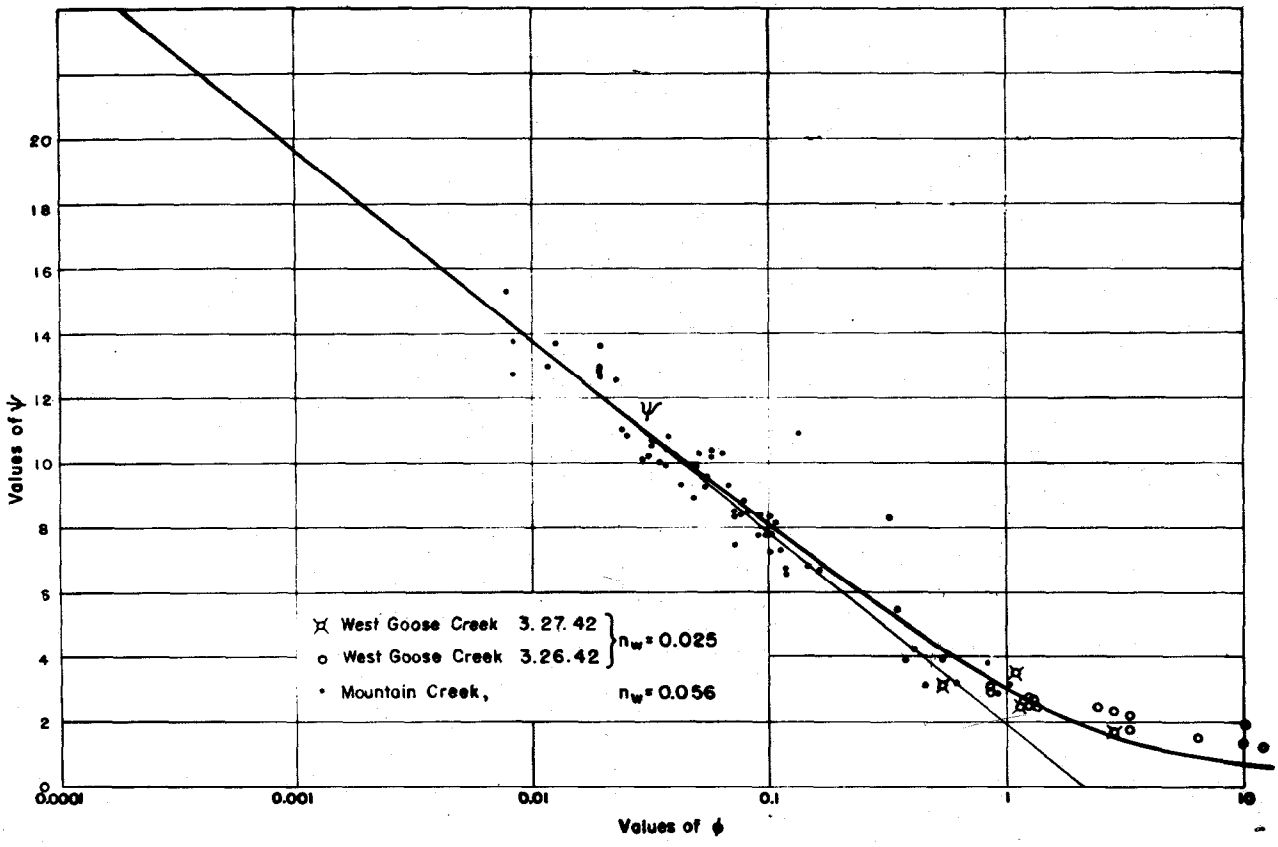


Fig. 5. Bed-load measurements in two streams plotted according to Einstein ϕ -ψ formula. (Reprinted from "Bed-load Transportation in Mountain Creek" (See "References," item 15, at close of comments)

In order to get a better idea of the size of streams covered by the ϕ -ψ graph, some numerical values will be substituted in the formula. By re-arranging the formula for ψ, the value of the hydraulic radius becomes

$$R = \frac{p_1 - \rho}{p} \frac{D}{\psi s} \qquad (14)$$

If we now assume a ψ value of 4, quartz sand with diameters of 1 and 0.5 mm., and channel slopes of .001 and .0001, we obtain values of R shown in Table 2.

TABLE 2. VALUES OF HYDRAULIC RADIUS FOR ψ = 4

for assumed values of D and S

D (mm)	D (ft)	S	R (ft)
1.0	.00326	.001	1.3
		.0001	13
0.5	.00163	.001	0.65
		.0001	6.5

15/ See "References," at close of comments.

With 1 mm. sand and a slope of .001, the hydraulic radius for a value of $\psi = 4$ will be 1.3 feet. With the smaller slope this value will go up to 13 feet. This would be a condition for a relatively small stream, but would still cover many conditions of practical interest. By going to lower values of ψ and higher values of ϕ on the graph, conditions for still larger streams would be covered.

In order to use any bed-load formula, one must be able to calculate the hydraulic radius from the discharge, or conversely, one must be able to calculate the discharge from the hydraulic radius. This means that we must know the roughness of the channel. A number of experimenters have reported that the roughness of a sand bed varies with the rate of transportation. In Fig. 6 Einstein 16/ has plotted a factor Γ, against ϕ, where Γ is the ratio of the Manning roughness, n_s for no transportation to the roughness, n, at any given rate of transportation. The value of the roughness, n_s, is given by the Strickler formula

$$n_s = 0.0132\, D_1^{1/6} \qquad (15)$$

where D_1 is a representative diameter of the sediment in mm. Einstein found that D_1 was the value for which 65 per cent of the material was smaller. With the above formulas, a complete calculation can be made of the bed-load transportation of a stream. It is the only complete bed-load equation available in the literature that is based on a modern analytical development. Kalinske 17/ has developed a formula based on statistical conceptions similar to those of Einstein. However, it involves quantities not measured in any experiments and its validity, therefore, remains to be proved. Recently Einstein 18/ has developed a method of calculating the load when an appreciable part of the sediment moves in suspension by dividing the load into bed and suspended load. Other workers 19/20/21/22/ have studied the forces acting on the sediment particles both experimentally and theoretically. Those were not intended to yield transportation formulas but to clarify the mechanism of movement which ultimately must form the basis for the formulas.

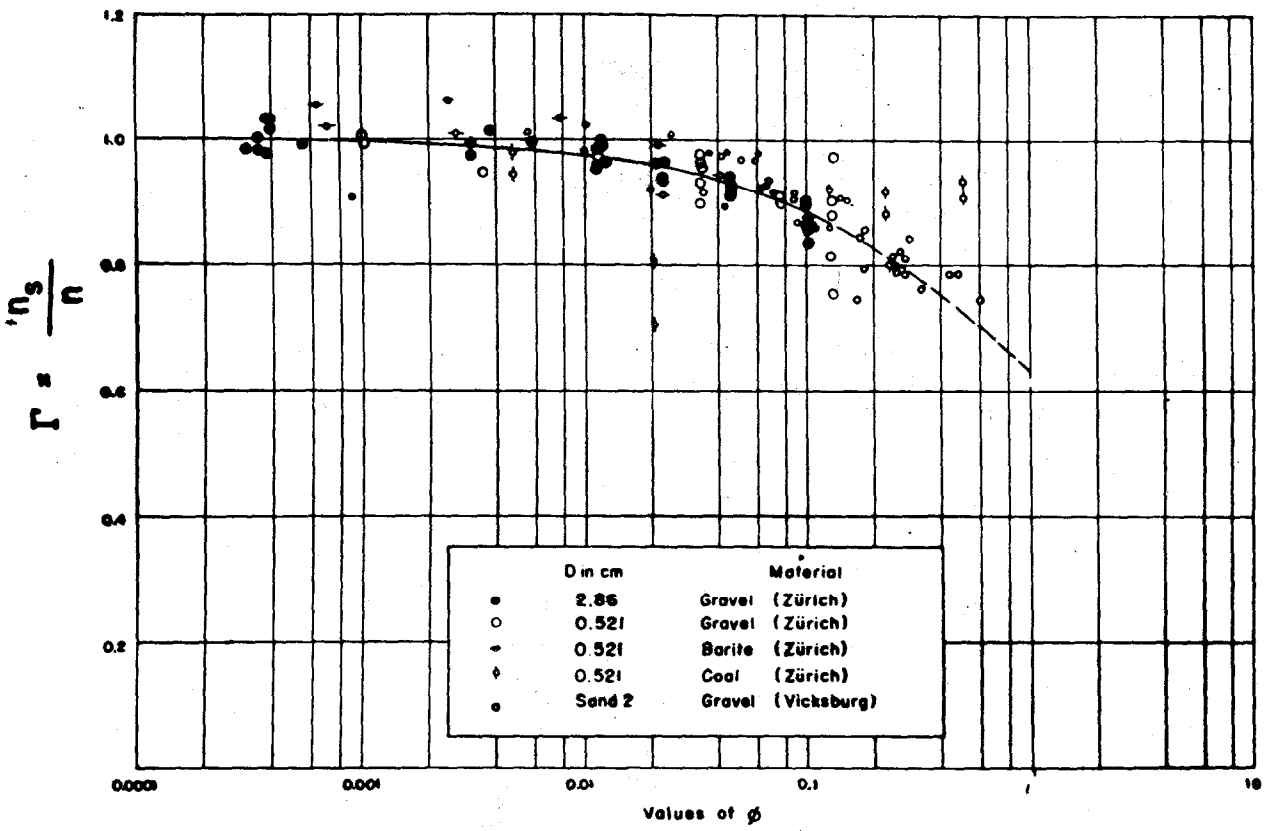


Fig. 6. Relative bed roughness plotted against ϕ according to Einstein. (Reprinted from "Flow on a Movable Bed" (See "References," item 16, at close of comments)

16/17/18/19/20/21/22/ See "References" at close of comments.

Suspended Load

The very rapid development of the knowledge of transportation of suspended sediment by flowing fluids is attributable directly to the development in the general field of fluid mechanics which was stimulated principally by the development of aeronautics starting at about the time of World War I. The basic concept of turbulence suspension of sediment was advanced by Wilhelm Schmidt 23/ in 1925 for the suspension of dust in air. In 1933 Leighly 24/ applied this method to the suspension of sediment in streams. In 1933 O'Brien 25/ derived the differential equation for distribution of suspended load in a stream, which is

$$w c + \epsilon \frac{dc}{dy} = 0 \qquad (16)$$

where w is the settling velocity of the particle in the still fluid, c is the concentration at a distance, y from the bottom, and ϵ is the so-called "exchange coefficient" which has the dimension of velocity times length. The left-hand term of the equation is merely the mass rate of settling of the particles under the force of gravity. The right-hand term is the rate of upward movement due to turbulence mixing. This work of O'Brien's set off a rapid sequence of developments which, in a period of a relatively few years, has developed the knowledge of suspended material to a degree comparable, if not in excess of that attained by the bed-load work in a period of about 60 years. O'Brien pointed out that the quantity, ϵ can be determined for momentum transfer from the velocity distribution by the formula,

$$\tau = \rho \epsilon \frac{dV}{dy} \qquad (17)$$

where τ is the shear stress and V is the velocity at a distance y from the bottom. If ϵ is constant then by integrating equation (16) we get the equation

$$c = c_a e^{-\frac{w}{\epsilon}(y - a)} \qquad (18)$$

where C_a is the concentration at some arbitrary level distant, a, from the bottom. This equation was checked by Hurst 26/ and Rouse 27/ for steady conditions with artifically-produced turbulence which was uniform over the depth, thus giving the condition of ϵ = constant. Dobbins 28/ has checked the relation for unsteady conditions with uniform turbulence. By assuming the von Karman logarithmic velocity distribution law,

$$\frac{V - V_{max}}{\sqrt{\frac{\tau_o}{\rho}}} = \frac{2.3}{k} \log \frac{y}{d} \qquad (19)$$

ϵ can be calculated from eq.(17) and eq. (16) can be integrated to give the sediment distribution in a 2-dimensional uniform flow of depth d. The result of this integration is given by

$$\frac{C}{C_a} = \left[\frac{d-y}{y} \times \frac{a}{d-a}\right]^z \qquad (20)$$

where $$z = \frac{w}{k\sqrt{gdS}}$$

k is the von Karman universal constant for turbulent flow which is about 0.4 for clear fluids, g is the acceleration of gravity, and S is the slope of the stream.

This equation was first published by Rouse 29/ in 1936. Fig. 7 is a graph of this equation for a number of assumed values of z. It will be noted that as z becomes small, the concentration tends to become more nearly uniform, while for high values of z the concentration vanishes in the upper part of the stream and most of the material is carried near the bed. It must be pointed out that this equation assumes that the transfer coefficient, ϵ, for momentum is the same as that for sediment. This is not necessarily so, as experiments have shown. Equation (16) was checked by Christiansen 30/ by applying it to field measurements of suspended load in the Imperial Valley Irrigation Canals. The form of the equation was checked by Vanoni 31/ in the laboratory experiments. The results of some of these experiments are shown in Fig. 8, in which measured concentration is plotted against depth. The curve at the right is for uniform material of 0.1 mm. diameter and the measurements follow a curve with value of the exponent z_1 of 0.34. The value of the exponent calculated from the formula is 0.423 and gives the dotted curve in Fig. 8.

23/24/25/26/27/28/29/30/ See "References" at close of comments.

In this case the measured concentration was actually more uniform than the concentrations predicted from the theory. (A similar result was found by Anderson 32/ for distribution of suspended load in a natural stream). The curve at the left is for a uniform material of 0.16 mm. diameter, and in this case, the value of the measured and calculated exponent agree very well. These results show the trend found in the experiments and indicate that although the form of the equation is correct, the constants given by the theory are in error.

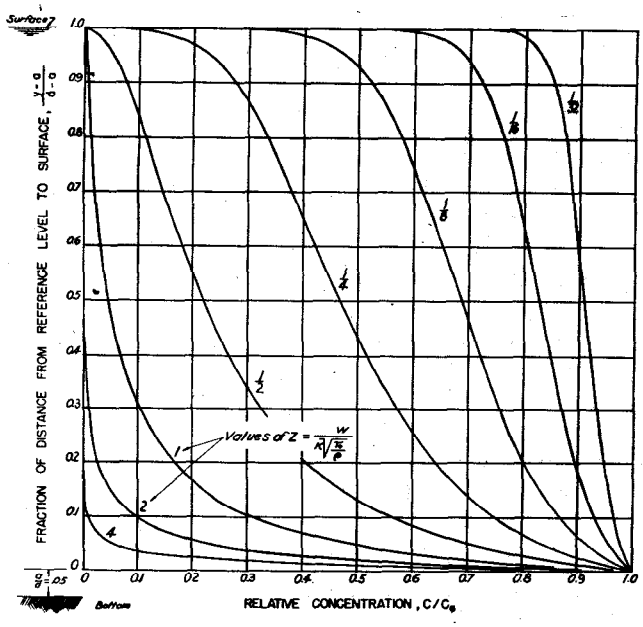


Fig. 7. Graph of theoretical suspended load distribution equation No. 20.

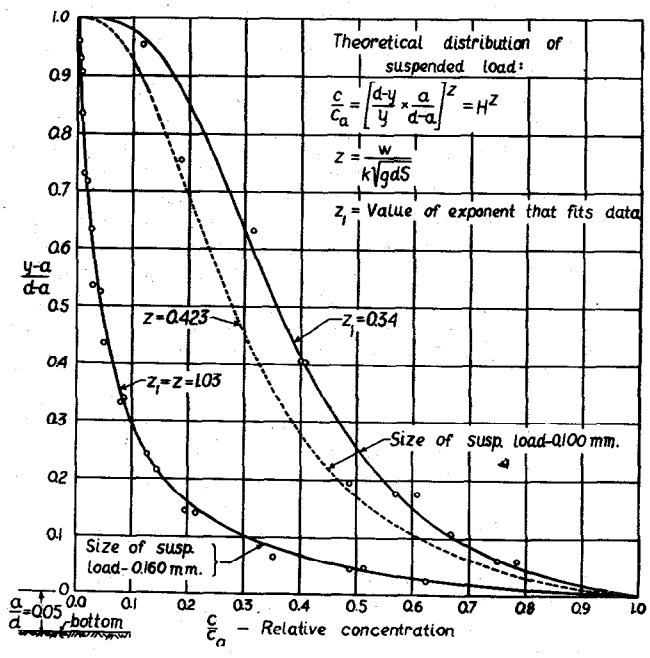


Fig. 8 - Comparison of theoretical and measured suspended load distribution for two sizes of sediment.

A very significant observation during the experiments was that sediment in suspension tended to cause an increase in the average velocity. This fact was attributed to the reduction in intensity of the turbulence by the suspended sediment. The energy to support the sediment must come from the turbulence and by giving up its energy the turbulence is less effective in mixing or agitating the flow. This is the same as saying that the transfer coefficient ϵ is reduced. From equation (17) it follows that if τ is kept constant as the sediment load increases, the value of the velocity gradient, and hence of the velocity, must increase. Careful velocity distribution measurements confirmed this. As the load increased, the slope of the velocity distribution curve also increased, tending to give greater differences in velocity between the bottom and the top. The amount of this increase can be expressed by the factor, k, in equation (19), since it was found that the velocity distribution equation even for relatively high loads still kept the logarithmic form of equation (19). Another factor that was found to vary with the amount of the load was the exponent, z. Fig. 9 is a graph of the three quantities, z, k and the resulting roughness coefficient, n, plotted as functions of the mean sediment concentration for 0.1 mm. sand. It will be seen that the roughness coefficient and the quantity, k, both are reduced as the concentration increases. The value of the exponent, on the other hand, increases as the sediment concentration increases. It will be remembered from Fig. 7 that as z increases, the distribution of material in the flow tends to become less uniform. Fig. 9, then, indicates that as the load increases, the sediment behaves as a coarser sediment, or, in other words, the ability of the stream to support it is lessened.

Another series of pertinent observations was that of the behavior of sediment in the flume cross section as a whole. The material tended to travel in bands or ribbons rather than uniformly, leading one to believe that there were certain secondary circulations present. No detail study was made of these, but there is some evidence indicating that it may be an important factor in the phenomenon. If so, the problem is much more complicated than had been anticipated since the

32/ See "References" at close of comments.

shape of the channel and the distribution of roughness over the wetted perimeter must certainly be important factors in the determination of secondary circulations.

It must be pointed out that the sediment distribution, equation (20), gives only the relative concentration and cannot be used to determine the load unless the concentration, C_a, at some level is known. Lane and Kalinske 33/ have developed an expression for the concentration just above the bed in terms of the properties of the sediment at the bed and a normal distribution of turbulence fluctuations. This is a step in the direction of calculating the total load of a stream in terms of the hydraulic factors. The trend in recent years has been to make less and less distinction between the material carried near the bed and that carried in suspension. In the opinion of the writer, this is a very sound trend since there does not appear to be any analytical reason for dividing the sediment transportation of a stream into the bed-load and the suspended load. The work of Einstein, Anderson, and Johnson 34/ in classifying the load of a stream into the "washload" and the "bed material load" has done much to bring this about. These writers defined the washload as a very fine material that is carried in suspension and that does not appear in the bed in appreciable quantities. They pointed out that this material is usually not a function of the stage, but depends on other conditions, such as the distribution of rainfall, condition of the watershed, etc. The bed material is that portion of the load having the same composition as the material in the bed. The rate of transportation of this material is a function of the stage.

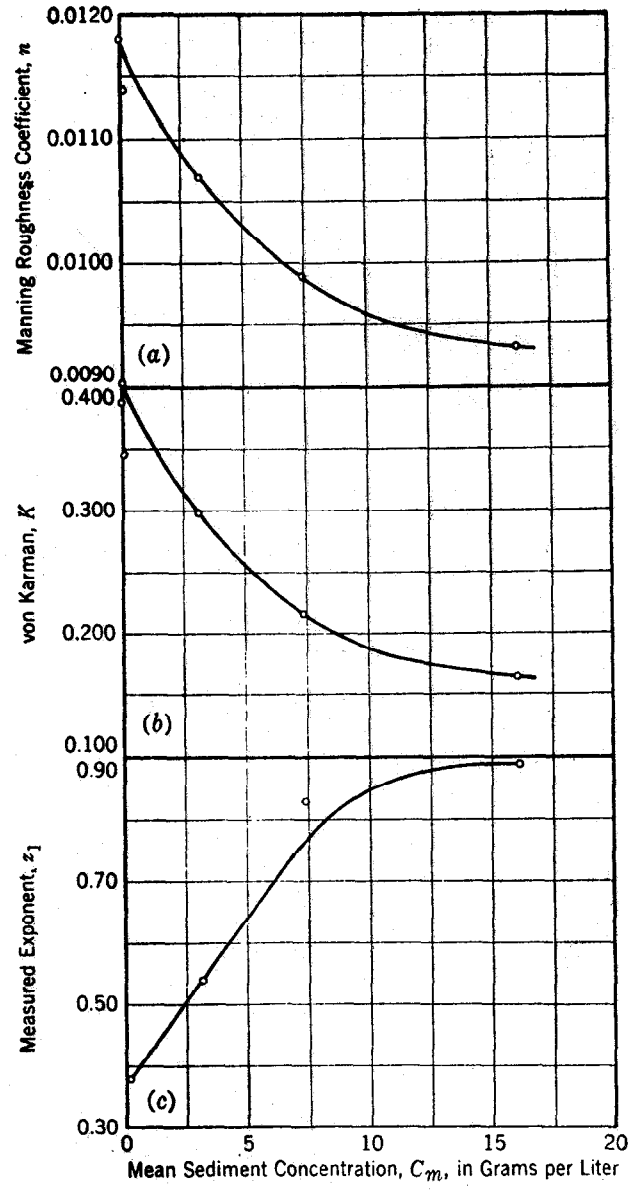


Fig. 9. Variation of apparent roughness , von Karman k and exponent Z_1 in sediment distribution equation with mean concentration. (Reprinted from "Transportation of Suspended Sediment by Water") (See "References," item 31, at close of comments).

Conclusions

One of the most important questions to be answered by a discussion of this kind is "Do we have a sediment transportation formula that can be applied to natural streams?" The only formula that is complete enough to be considered in such a discussion is the formula of Einstein. It agrees with a wide range of laboratory experiments from a number of sources and has also been found to agree with field measurements in small streams. The range covered by the formula up to $\phi = 10$ does not cover conditions where the rates of transportation are relatively high, such as in many of our streams during flood stage. This formula has not actually been tried on larger streams and therefore has not proved itself completely. However, it shows good promise and can at least serve to guide engineers in planning works where sediment transportation is involved. Formulas of the DuBoys type apparently fit only a very narrow range of conditions. They can probably be used as interpolation formulas on particular streams provided one can determine the constants by a combination of laboratory and field measurements. However, it does not seem possible to use this type of formula to calculate sediment loads without making elaborate field checks.

33/ 34/ See "References" at close of comments.

It is clear from this discussion that the sediment transportation problem is far from being solved. The fact that many able investigators have devoted much time to it without making more progress is evidence that the problem is complicated. Therefore, it is one that requires continued undivided effort on the part of the able investigators if we are to progress with it. The economic importance of the problem needs no emphasis. This has already been done by the elaborate programs of development on our major streams involving the expenditure of many millions of dollars. If we are to do a creditable job on this development, then we must learn more about sediment transportation and the mechanics of streams in general.

The following is a list of the problems on sediment transportation that I feel are of importance and should be studied further.

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

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

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

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

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

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

As pointed out previously, such investigations must involve continued effort on the part of highly trained research men. I estimate that there are less than 10 professional men in this country devoting a major part of their time to the study of the mechanics of sediment transportation. This is far too small a number. In my estimation, the sediment problem certainly ranks in importance with the more popular new problems of atomic energy, rocket propulsion, and travel at supersonic speeds. Yet it is receiving a negligible amount of research effort compared to these problems. It is up to those of us that know the vital importance of the sediment problem to call it to the attention of the people so its study will receive adequate financial support and the attention of our many research engineers and scientists.

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DISCUSSION

M. A. MASON,* This summary of the development of our knowledge of the mechanics of sediment transportation needs no amplification of its technical aspects, but its philosophical aspects can well be considered.

Recall that DuBoys classic work was the outgrowth of a study of natural river problems, specifically a question of the quantity of material carried by the Rhone River. Today we find that despite the advances made since DuBoys, the same question cannot be answered without reservation. Dr. Vanoni attributes this condition in part to the complexity of the problem, and in part to the small number of competent workers in the field. I believe a third factor, presently of transcendent importance, should be considered. That factor is our apparent inability to relate the fundamental technical aspects of the problem to our common economy in such a way that the financial support required to prosecute needed research and studies is available and assured.

We, as investigators or technical administrators, are all familiar with this phase of the problem. We are all in accord, I believe, that the technical problem involved could be solved if adequate financial support for a well-conceived overall program of study were assured. But have we prepared such a well-conceived overall program? Have we thoroughly examined, even in our own minds, the justification for the expenditure of funds, either private or public, on such a program? We must admit negative replies to these questions - we must admit, also, our frequent inability to convince even our own departmental budget and administrative officials of the necessity for our own limited programs. This is a deficiency in the leadership that can be reasonably expected of engineers. It is a deficiency that can be supplied by the Inter-Agency

* The Beach Erosion Board, Corps of Engineers, Department of the Army, Washington, D. C.

River Basin Committee, by the professional societies, even by a group of workers in the field. Certainly it is a deficiency that, under present conditions, is a major contributing cause to the difficulty of further work in this field.

The formulation of a well-conceived program in sediment transportation research is by no means a simple task. Effort in the past, with a few notable exceptions, has been concentrated on the practical aspects of the river and stream, bed and suspended load transportation problems; but the whole field is much broader. The list furnished by Dr. Vanoni has not exhausted his own possibilities, and each of us could add fundamental items. In the study of shore phenomena, for example, sediment, or more accurately, material, transportation is a major concern. Sediment is carried by streams to the sea as an ultimate destination; arriving there it is subjected to new transportation processes. Littoral movement by water forces and aeolian transportation are the more important processes. In each of these the relatively simple conditions of stream transportation are complicated enormously by the variety and nature of the forms of energy active at the shore. Only in a shore environment can we find material movement governed by the extreme violence of a breaking wave associated intimately with the turbulent upsurge and super-critical return flow down the beach of the same wave. Superimpose on this picture littoral currents resulting from tidal or wind effects and the composition is complicated indeed. Yet this is a legitimate and economically important sediment transportation problem. At the other extreme we can consider the transportation effects of ordinary rainfall on a vegetated area. Wind transportation phenomena must be included in our program, the change in fluid medium from water to air being a legitimate variation that our finally developed mechanics knowledge should be capable of handling.

Is it not reasonable to think of the common fundamental aspects of each of these varied yet related problems gathered into a program of basic study and research jointly financed and prosecuted? With this basic data in hand will not the long periods of comparative sterility between DuBoys, Gilbert, and Einstein be avoided? Will there not be opportunity for a more logical and successful appeal for financial support from government, university, and industrial funds?

Perhaps this latter question involves a realism that has no place in a technical discussion. But the record presented by Dr. Vanoni indicated to my mind that the deficiencies in our knowledge of the mechanics of sediment transportation could be attributed more to lack of specification of what we need to know and of support in satisfying our needs than to lack of technical competence in solving the problems once adequately defined. If this is true then further advances by other than a system of chance are contingent on supplying these past and present deficiencies. To my mind this can be done best and only by a wise, realistic recognition that what we have been doing is not enough, and by translating that recognition into a deliberate effort to actively endow critically needed basic research.

PARKER D. TRASK.* Mr. Vanoni's paper is on a subject of fundamental importance to everybody who works in the field of sedimentation. We cannot hope to solve most of our problems until we know the basic laws governing the erosion, transport, and deposition of sediments. Most of us are concerned with practical applications of these laws and we make the best approach we can by means of empirical data; but if we knew how the fundamental factors behaved and the inter-relationships of these factors one with another, we could predict what will happen with much greater certainty than we can now.

Messrs. Vanoni, Einstein, Lane, Johnson, Sverdrup, Monck, and the handful of others who focus their attention on these natural laws should be encouraged to continue their work, not only with funds and facilities, but also by the participation of others in this field.

H. A. EINSTEIN.** I thought you might be interested to know, in a general way at least, in which direction research in this field seems to move. Maybe I can make a few remarks about that. You have seen from Dr. Vanoni's paper that there exist essentially two separate theories: one for bed load and one for suspended load. The bed-load theory tries to give the rate at which sediment is transported under certain given flow conditions. The suspended-load theory tries to do something entirely different. It doesn't give the rate itself but it gives the distribution of concentrations as ratios over the different locations in the flow. It has been found and brought out, especially by Professor Lane, who unfortunately isn't here, that also for suspended loads

* The Geological Survey, Washington, D. C.

** Cooperative Laboratory, Soil Conservation Service and California Institute of Technology, Pasadena, Calif.

there exist in the river certain laws according to which, at least for the coarser particles, the absolute value of the concentration is a function of the flow. If such a relationship exists in nature then we must be able to find why it exists and must be able to predict it. It has been in this direction that the latest endeavors have pointed. I can't describe here in detail how that was done, but the approach is based on the general idea that the rate at which sediment moves right at the bed can be given under all conditions by what we usually call a bed load formula. This bed load moving right at the bed in a layer about 2 grain diameters thick represents a sediment concentration right there near the bed from where we are able then to calculate the concentrations higher up by means of the suspended-load theory. In other words, the two types of approach can be combined with comparative ease into one unit.

A big difficulty that we still encounter in trying to predict the transport in rivers is based on the fact that we do not know enough about the behavior of different sized particles within a mixture. It has been possible in many cases to assume that the different particles behave very much alike or, in other words, that we find the same mixture in the bed and in movement. As soon as we combine into one unit the two theories of bed load and suspended load that assumption doesn't hold at all any more, and then we must replace it by another one. The new assumption stipulates that the movement of any one particle is practically independent from the movement of the other particles and may be fully described as a function of the flow. Now, I can't very well elaborate on the way this assumption works out, but it is a very reasonable one and seems to give good results. I don't think that it will be very long until we are able to really calculate the transport of material which moves partly in suspension and partly as bed load, which is the normal case in most of our Western streams. I believe if we do as the other speakers mentioned, that is, if we try to impress those agencies and people who are responsible for the distribution of funds and induce them to spend just a small part of the total research funds on this problem, we should be able pretty soon to solve a great number of practical problems by means of a theoretical approach.

APPLICABILITY OF MODEL STUDIES TO SEDIMENT PROBLEMS IN NAVIGATION CHANNELS

by H. B. Simmons, G. B. Fenwick, and C. B. Patterson. *

The reason for the "Navigation Channels" limitation in the title of this paper is obvious, since the Corps of Engineers, being responsible for the development and maintenance of the navigable rivers and harbors of our country, is deeply concerned with the reduction of sedimentation, or shoaling, of such channels. Reduction in shoaling of artifically maintained channels means reduction in the cost of maintenance, and increase in efficiency of the channels as navigable routes. The work of the Waterways Experiment Station in this particular respect has been confined to studies of specific sedimentation problems, with a view toward designing and developing plans of improvement for the reduction or elimination of shoaling.

Sedimentation problems which have been investigated by means of hydraulic models at the Waterways Experiment Station can be roughly divided into two groups, the first group consisting of problems caused by bed-load movement, and the second of problems caused by deposition of suspended silt. In certain prototype sedimentation problems, it has been found that both bed-load movement and deposition of suspended silt contribute to shoaling of navigation channels. Problems caused by bed-load movement may be found in streams with stable banks as well as in streams with easily eroded banks, and deposition problems may be found in fresh water rivers and harbors as well as in salt water rivers and harbors. It is evident, therefore, that the various types of sedimentation, or combinations of types, which contribute to channel shoaling are numerous.

The two basic types of models used for investigations of sedimentation problems are the movable-bed model and the fixed-bed model. For either type, a relatively small section of a river, or an entire tidal estuary, may be reproduced in the model as required by the specific problem under study. The movable-bed model is constructed with a movable-bed section of sand, crushed coal, haydite, or other movable material, and the banks are either of concrete or erodible materials, dependent upon the type of problem under investigation. Prototype bed movement in this type model is reproduced by adjusting the hydraulic forces in the model and altering the characteristics of the movable-bed material until the model will reproduce, within specific time intervals, changes in bed configuration which are known to have occurred in nature. The fixed-bed model is constructed entirely of concrete or other stable materials. Sedimentation in the fixed-bed model is reproduced by injecting finely-ground, light-weight shoaling material into the model in such a manner that prototype rates and/or distributions of shoaling are reproduced accurately. The fixed-bed model is also used for investigations of sedimentation problems through a comprehensive study of the effects of proposed improvement plans on tides, current velocities and directions, and salinity. In the latter type of study, no attempt is made to reproduce sedimentation in the model; however, the effects of proposed improvement plans on sedimentation in nature can be evaluated by an analysis of the hydraulic effects of such plans in the model.

The applicability of hydraulic model studies to the general types of prototype sediment problems mentioned previously can best be demonstrated through discussion of specific model studies. Since space will not allow for discussion of all of the various model types which have been and are being utilized, typical examples of three general types have been selected: (a) a problem caused by bed-load movement in a stream with stable banks; (b) a problem caused by deposition of suspended silt in a tidal stream and, to a lesser extent, the movement of deposited silt along the bed of the stream; and (c) a problem caused by deposition of suspended silt in a tidal stream, in which salinity and salt-water currents had a definite relation to the problem.

As a typical example of model studies of sedimentation problems arising from bed-load movement in streams with stable banks, a model study of the Pryors Island reach of the Ohio River has been selected (Figure 1). In the maintenance of a 9-foot navigation channel through this

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* Chief, Estuaries Section, Hydraulics Division, Chief, Rivers and Harbors Branch, Hydraulics Division, and Chief, Research Center, respectively, Waterways Experiment Station, Corps of Engineers, Department of the Army, Vicksburg, Miss. (Presented by H. B. Simmons).

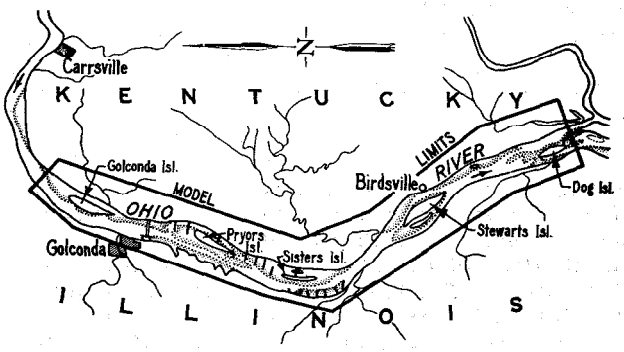


Figure 1. PRYORS ISLAND MODEL STUDY LOCATION MAP

reach, objectionable shoaling is encountered at three localities: from the head of Pryors Island to the foot of Sisters Island; just above Stewarts Island; and at Old Maids Crossing just above Dog Island. The purpose of the model study was to develop plans to alleviate this condition.

The model was a reproduction of the area shown, to linear scale ratios of 1:600 horizontally and 1:150 vertically. Banks and overbank areas were molded in concrete to prototype configurations. The stream bed was of crushed coal, molded to prototype contours by means of removable templets. Provisions were made for reproducing any desired discharge hydrograph and for setting proper flow lines in the model. The operation of the model was divided into two distinct phases: first, the adjustment of the model and the verification of its accuracy; second, the developing and testing of improvement plans for the solution of the shoaling problem.

The verification of a movable-bed model is a highly important phase of such a study, since the validity of the entire study rests upon the results of this procedure. The computation of the proper time scale and certain other scale relationships in such a model is impossible, due largely to the fact that mathematically determined relationships fail completely because practical considerations do not permit the application of such relationships to available and usable movable-bed material. Hence, the model verification -- which might be termed the empirical determination of scale relationships -- must be depended upon to establish the applicability of the model results to the prototype.

Verification of the Pryors Island model consisted of a series of tests during which the time scale, slope, bed load, and roughness were progressively adjusted by trial-and-error until the model would reproduce accurately the hydrographic changes which were known to have occurred in the river between two surveys made five years apart. This was accomplished by molding the movable bed to the configuration shown by the earlier survey, and then reproducing in the model the sequence of stages which occurred in the river during the five-year period, all artificial changes such as dike construction and dredging being reproduced in chronological order. The model verification was considered satisfactory when a survey of the movable bed, made at the end of this operation, showed the bed configuration to be an accurate reproduction of the river conditions shown by the later survey.

The model verification fixed the procedure for all subsequent tests of possible improvement plans, and this procedure was maintained exactly during all tests. Thus, the results of all tests were directly comparable with each other. In all, twenty different plans or modifications were tested in the model in arriving at one which would alleviate the shoaling problem in this reach of the river. The river survey of 1930 (Figure 2) shows intermittent shoals from above Pryors Island to below Sisters Island, a shoal above Stewarts Island, and a dangerous crossing above Dog Island. The general plan of improvement for this reach involved channel realignment and the construction of stone spur dikes.

Figure 3 is the recommended plan of improvement which was developed in the model. The Pryors and Sisters Island back channels were more effectively diked and the existing dikes

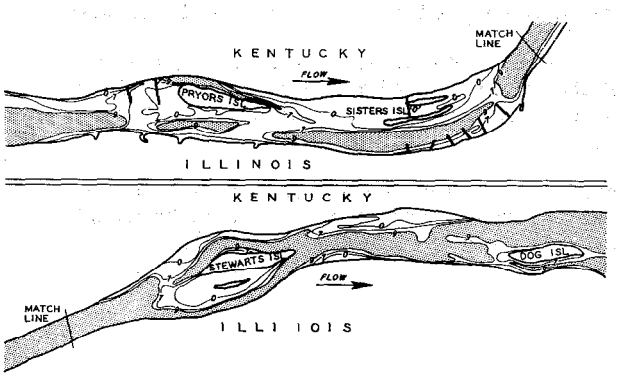


Figure 2. MODEL STUDY OF PRYORS ISLAND REACH OHIO RIVER RIVER SURVEY OF 1930

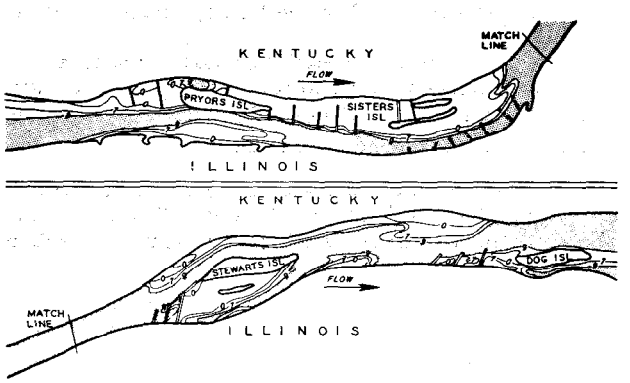


Figure 3. MODEL STUDY OF PRYORS ISLAND REACH OHIO RIVER RECOMMENDED PLAN

between these islands were raised and extended. The channel opposite Pryors Island was shifted from the right bank to the left by dredging to a 12-foot depth at the beginning of the test. The seven spur dikes opposite Sisters Island were completely removed. A group of spur dikes was constructed above Stewarts Island and another group above Dog Island. No dredging was done in the model except during the first of the seven years reproduced in this test, yet a 9-foot navigable channel was open at the end of the test and the bed had reached a stable condition.

As an example of model studies of sedimentation problems caused by deposition of suspended silt, a model study of Deepwater Point Range of the Delaware River has been selected. Deepwater Point Range is approximately 4.5 miles in length and 800 feet wide with a project depth of 40 feet. At the time of the model study, the range shoaled at an average rate of approximately 2,800,000 cubic yards annually, requiring almost continuous dredging to maintain project depth. The heavy shoaling was due, according to extensive studies, to lack of parallelism between the currents and the channel.

The model in which the Deepwater Point Range study was conducted reproduced that portion of the Delaware River from Artificial Island on the downstream end to Bellevue on the upstream end, and a short section of the Chesapeake and Delaware Canal (see Figure 4). Tides were controlled by automatic tide reproducers at the two ends of the model and in the Chesapeake and Delaware Canal, and the roughness value of the model channel was so adjusted that tides and current velocities and directions were reproduced accurately throughout the model. Shoaling was reproduced by injecting into the model measured volumes of a mixture of water and finely-ground gilsonite, which is a light-weight bituminous material having a specific gravity of about 1.035. The shoaling material was injected into the model upstream from the problem area during the ebb-tide period and downstream from the problem area during the flood-tide period for several successive tidal cycles. The gilsonite moved from the points of injection to and throughout the problem area in suspension and in some degree along the bed; that is, most of the silt was deposited directly in the problem area, while a smaller amount was deposited elsewhere and moved into the problem area by rolling along the model bed. Following completion of a shoaling test on the model, the model was pooled and the gilsonite which had deposited in the channels was picked up and measured. No attempt was made to reproduce volumes of prototype shoaling, since formation of deep deposits of gilsonite on the concrete bed of the model in areas exposed to the tidal currents would have been impossible. Instead, the characteristics of the mixture and the method of introducing the mixture into the model were varied through a cut-and-try process until the distribution of shoaling material throughout the problem area conformed to the measured distribution of shoaling in the prototype. The operating procedure developed during adjustment of the model -- the gilsonite-water mixture used, the volume introduced, the method of introduction, and the exact time of operation of the model after the material had been introduced -- was followed strictly during all tests of improvement plans.

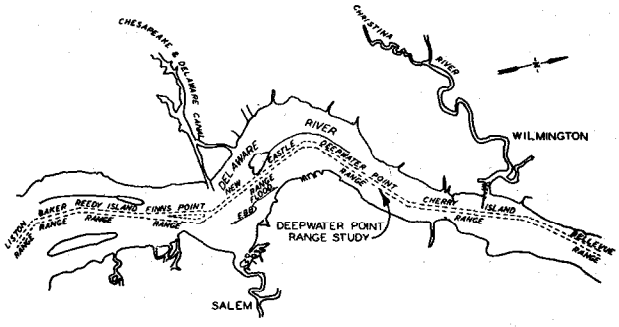
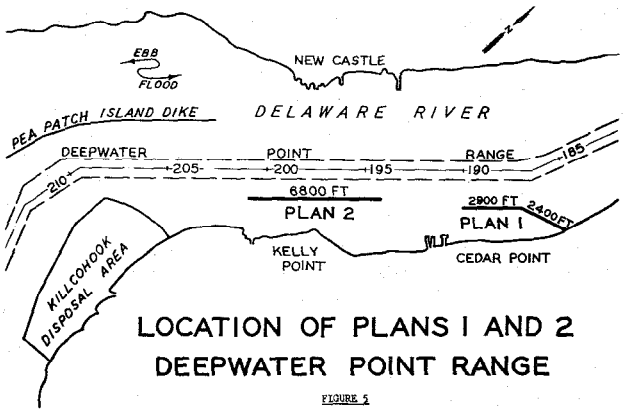


Figure 4. MODEL STUDY OF DEEPWATER POINT RANGE, DELAWARE RIVER

In order to evaluate correctly the effects of each proposed improvement plan on shoaling of the channels, a base test, or test of existing prototype conditions, was first established, and the measured amount of shoaling obtained during each test of an improvement plan was compared to that obtained during the base test. The effect of each plan was then determined in terms of percentile reduction or percentile increase in shoaling as compared to the base test.

As previously stated, the heavy shoaling in Deepwater Point Range was attributed to lack of parallelism between the currents and the channel. Therefore, plans for the improvement of this condition were designed either to realign the currents to confrom with the channel alignment or, conversely, to realign the channel to conform with existing current alignments. Numerous plans of each type were designed and tested, and it was found that two plans, both designed to realign the currents to conform with the channel alignment, indicated reductions in shoaling of approximately 50 per cent. Plan 1, which indicated a reduction in shoaling of 47 per cent, consisted of a dike beginning at a point 1,900 feet east of the center line of Deepwater Point Range, opposite channel station 189+500, extending upstream parallel to the channel for 2,900 feet, thence forming an angle of 151 degrees and extending for 2,400 feet and tying in to the shore (Figure 5). The top elevation of the dike was approximately 2 feet above mean high water. Plan 2, which indicated



LOCATION OF PLANS I AND 2
DEEPWATER POINT RANGE

FIGURE 5

a reduction in shoaling of 59 per cent, consisted of a straight, 6,800-foot dike, beginning at a point 700 feet east of the channel center line opposite channel station 200+900, and extending upstream parallel to the channel to station 194+100 (Figure 5). The top elevation of this dike was also about 2 feet above mean high water. It first appeared that plan 2, which effected a slightly greater reduction in shoaling than did plan 1, would be the better plan. However, on closer examination, it was found that current patterns over the problem area as influenced by plan 2 were not so satisfactory as for plan 1 (see Figure 6). Furthermore, the plan 2 structure, being located in fairly deep water over its entire length, would be considerably more expensive to construct in nature than would plan 1; therefore, it was recommended that plan 1 be installed in the river.

Plan 1, now known as the Pennsville Dike, was constructed in the river during the period April 1942 to June 1943. A study of the effects of this dike on prototype shoaling during the period June 1943 to November 1946 has just been completed by the Philadelphia District Office of the Corps of Engineers. It was found that during this period, Deepwater Point Range shoaled at an

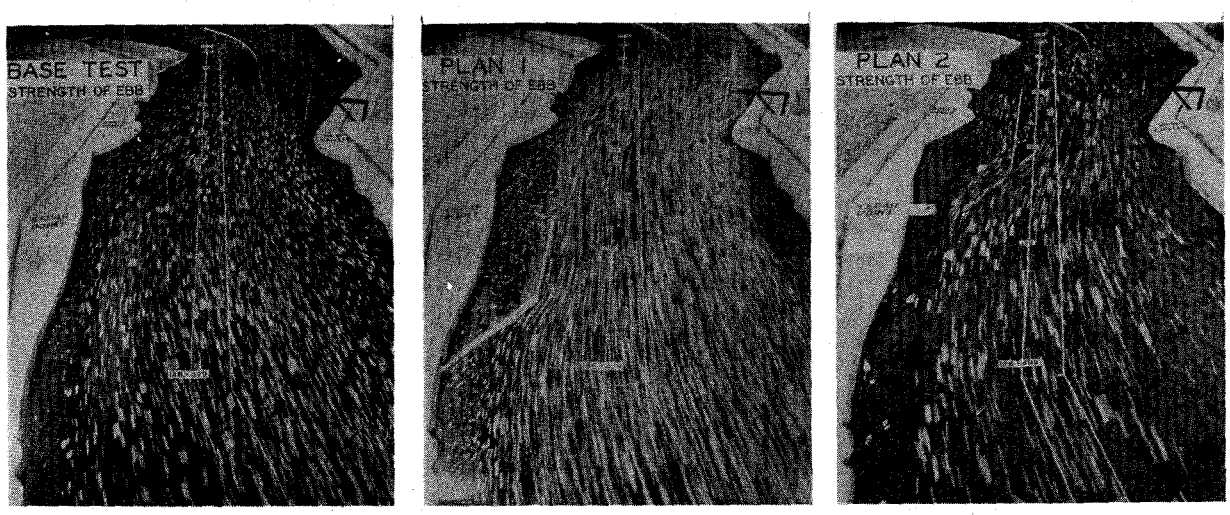


Figure 6.

average annual rate of 1,470,000 cubic yards, or a reduction in shoaling of 48 percent from the average rate before construction of the dike. The predicted decrease in shoaling for this structure, as obtained from the model study, was 47 percent. Construction of the dike effected some increase in the rate of shoaling in the lower end of Sherry Island Range, a short distance upstream from the Pennsville Dike; however, the increased shoaling in that locality is considerably less than one-half the reduction in Deepwater Point Range effected by the structure. Taking into account the initial cost of the structure, interest thereon, and the cost of maintenance, as against the cost of dredging material which would have deposited in Deepwater Point Range except for the dike, the actual monetary saving amounts to approximately $67,000 annually (Entire cost of model study was less than $12,000).

As a typical example of model studies of sedimentation problems caused by deposition of suspended silt, the results of which studies are based upon a comprehensive study of tides, currents, and salinity instead of upon simulation and study of shoaling in the model, the model study of Savannah Harbor has been selected. Savannah Harbor comprises the lower 22 miles of the Savannah River and forms the navigable waterway from the port of Savannah, Georgia, to the Atlantic Ocean. The navigation channel is maintained to a depth of 30 feet below mean low water over a suitable width, and is subject to shoaling in several reaches of the harbor.

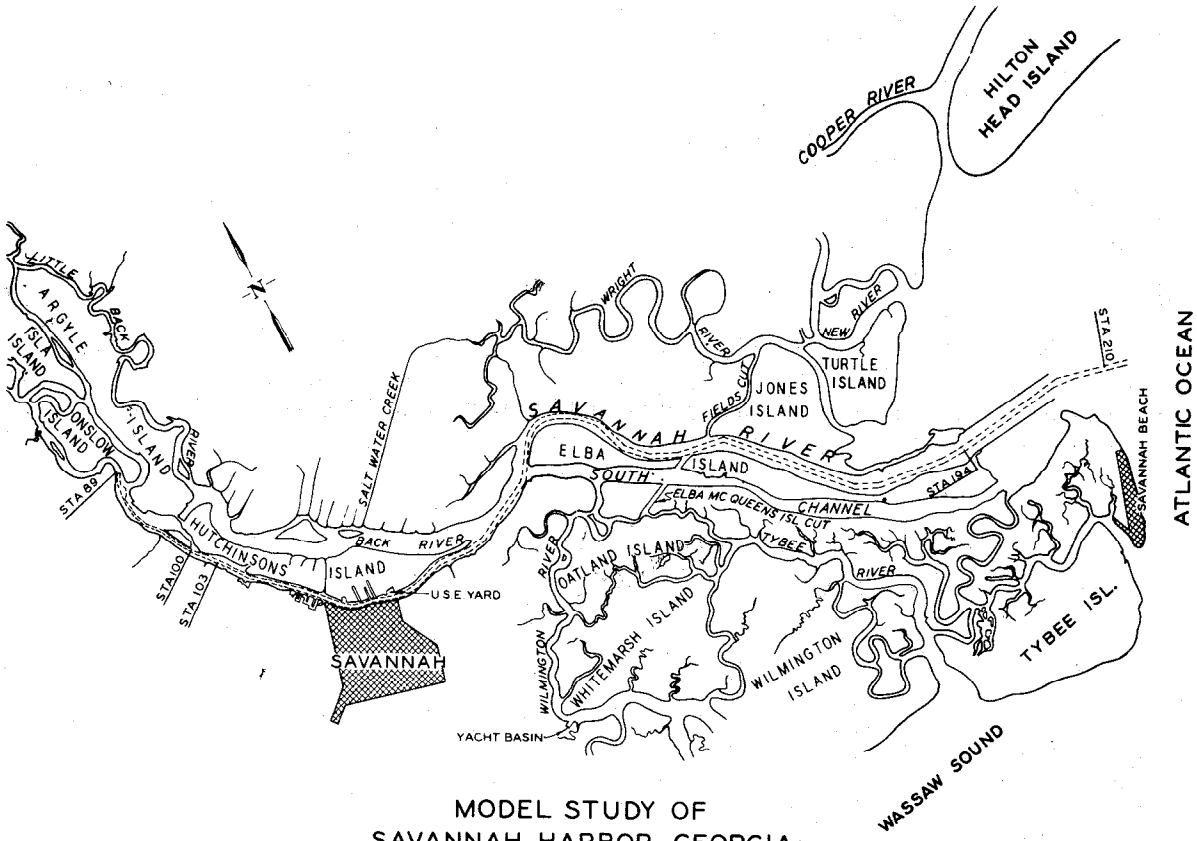
For the most part, the material which deposits in the navigation channel consists of finely divided clay particles which are brought down from the headwaters of the river by the fresh water flow. These very small and light particles will remain in suspension in fresh water with very little turbulence of flow; however, upon coming into contact with salt water, the silt particles undergo a process of flocculation, whereby the individual particles are attracted to each other and form into lumps of sufficient size and weight to fall to the bottom of the channel. One has only to see the Savannah River at the head of the project channel, where the color of the water is about that of tomato soup, and again at the mouth of the river where, for normal river discharges, the river water is almost as clear as that of the ocean, to understand fully the enormous volume of silt that is being deposited over the 22-mile length of the navigation channel.

The section of the harbor which is subject to heaviest shoaling is the lower 4-mile reach of Front River between the city of Savannah and the intersection of Back and Front Rivers. For the most part, shoaling in this reach occurs during late summer and fall, during which period the fresh water flow of the river is lowest and salinities in that portion of the harbor are highest. This shoaling is due to the fact that, during conditions of fresh water flow and salinity just mentioned, there is no appreciable ebb velocity along the bottom throughout the problem reach, while surface ebb velocities are about 2.5 to 3.0 feet per second. This is not the case during the flood-tide period, as bottom flood velocities are only slightly less than those at the surface. This unequal distribution of flow is brought about by the fact that Back River, which comprises a large percentage of the entire tidal prism lying above its confluence with Front River, carries no appreciable fresh water flow, but fills and empties only through its downstream extremity. Therefore, at high tide, the Back River tidal prism is filled with salt water of a fairly high specific gravity, since all water entering Back River during the flood period entered from the downstream end. During the ebb period, the flow of heavy salt water from Back River interferes with the ebb flow along the bottom in lower Front River to such extent that both ebb velocity and ebb duration are decreased considerably. Therefore, most of the shoaling material which deposits in the navigation channel above the mouth of Front River, plus some amount which deposits below this point and is brought upstream by the relatively strong flood currents, accumulates in the lower 4 miles of Front River from which it must be removed by dredging. Shoaling in this reach during periods of low fresh water flow is further aggravated by the fact that the salt water wedge intrudes further into the river, and flocculation of suspended silt occurs over a considerable distance upstream from the mouth of Front River.

The Savannah Harbor model, which was used for study of the problem just described and of several other problems, reproduced an area of the Atlantic Ocean from Wassaw Sound on the south to Hilton Head Island on the north and offshore to about the 40-foot contour of depth; the Savannah River from its mouth to the head of tide; and the maze of interconnecting channels tributary to the Savannah River (see Figure 7). The model was of fixed-bed construction, all channel and overbank areas being molded in concrete. Tides were reproduced in the simulated Atlantic Ocean and in Calibogue Sound, and the scale discharge of fresh water, representing the fresh water flow of the river, was introduced into the upstream end of the model. The salinity of the simulated ocean was maintained at the measured salinity of the prototype, so that density currents and their effects were reproduced accurately throughout the model. No attempt was made to reproduce or simulate prototype shoaling in the Savannah Harbor model; instead, a comprehensive study was made of the effects of each plan on hydraulic conditions in the harbor, and upon this basis the effects of the plan on prototype shoaling were predicted.

Plans designed and tested to reduce shoaling in lower Front River consisted of deepening, widening, and extending upstream the project channel, construction of submerged sills across the mouth of Back River, construction of a dam and tide gate in Back River and a dredge cut across Hutchinson Island above the city of Savannah, and replacing the tide gate of the latter-mentioned plan with a solid dam. Tests of widening, deepening, and otherwise improving the Front River project channel indicated that bottom ebb velocities in lower Front River could be increased appreciably by this method. As was expected, improvements in the hydraulic efficiency of the channel allowed the tide wave to run more freely, and thus effected increases in velocities throughout Front River. Tests of submerged sills across the mouth of Back River indicated that this method would have little, if any, effect on velocities in Front River. By far the greatest increase in ebb velocity in lower Front River was effected by the tide gate in Back River and the Hutchinson Island cut (Figure 8).

Operation of this plan was such that the tide gate opened during the flood-tide period and the flood tide entered Back River normally; the gate closed at high water slack; and, during the ebb-tide period, the Back River tidal prism above the gate emptied through the Hutchinson Island cut and thence through lower Front River, thereby increasing ebb velocities in lower Front River to a large extent. Tests of replacing the tide gate with a solid dam indicated that this method



MODEL STUDY OF
SAVANNAH HARBOR, GEORGIA

FIGURE 7

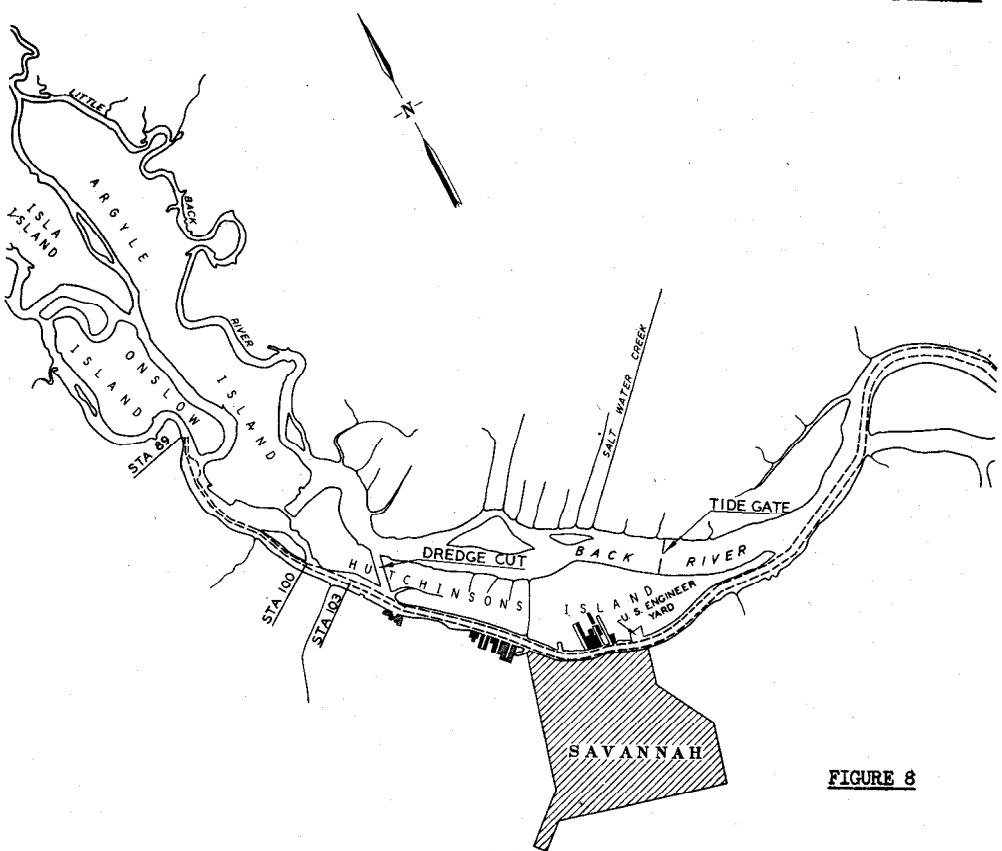


FIGURE 8

would also effect large increases in velocity in lower Front River; however, with the solid dam instead of the tide gate, both flood and ebb velocities in Front River were increased. By varying the location of the tide gate or the solid dam in Back River, thus varying the portion of the Back River tidal prism which would be added to Front River by either of these plans, it was found that velocities in lower Front River could be increased up to a maximum of 4 to 5 feet per second. It was also found, however, from a study of the effects of the tide gate and solid dam plans upon the harbor as a whole, that either of these methods which added a sufficient portion of the Back River tidal prism to that of Front River to increase appreciably the velocities in lower Front River would also have the effect of decreasing the tidal prism of the harbor. A decrease in tidal prism might cause shoaling in areas of the harbor not subject to shoaling at present, since such decrease would be accompanied by decreases in mean current velocities. Decreases in current velocities throughout the harbor would be proportional to the decrease in tidal prism. It was recommended, therefore, that it first be attempted to reduce shoaling in lower Front River by improving the hydraulic efficiency of the navigation channel and then, if additional works are required, to install one of the tide-gate or solid-dam plans which effected only a slight reduction in tidal prism of the harbor as a whole.

It has been demonstrated during this discussion that the adjustment and verification of the sedimentation model, and hence the accuracy of results to be obtained therefrom, are based upon data obtained from comprehensive prototype investigations. The completeness and accuracy of such prototype studies are most essential, since the model study would produce erroneous results if its adjustment and verification were based upon inaccurate or incomplete field data.

In the above connection, model-prototype confirmation studies are of inestimable value to the engineer who works with sedimentation models. Following the installation of an improvement plan in the prototype as a result of a sedimentation model study, the question immediately arises as to how closely the functioning of this plan in nature corresponds to the model predictions. Where inconsistencies are revealed through such a confirmation study, model operating techniques may be improved to the end of eliminating such inconsistencies in the future. Model-prototype confirmation studies are believed to be of such importance to the further development of sedimentation model techniques -- and thus to the solution of the problems involved -- that plans for a confirmation study should be included as a part of each comprehensive improvement plan that has involved study on a sedimentation model.

It must be understood that the hydraulic model is not proposed as a "cure-all" for sedimentation problems, nor does its use preclude the need for extensive field investigations in evolving plans for their solution. On the other hand, the sedimentation model, due to uncertainties which still exist as to the mechanics and principles of sedimentation in a prototype system, can be made to provide information not obtainable by other means. The need for experimentation is basically fostered by the fact that present-day knowledge of sedimentation phenomena and river hydraulics has not advanced to the point where such problems can be resolved by theory alone. And until that point is reached the model, with all its inherent limitations, will continue to be the most useful expedient.

A greater knowledge and a clearer understanding of the mechanics and basic principles of sediment movements have tremendous implication on the future progress of river and harbor engineering. The Corps of Engineers, participating in the program of the Federal Inter-Agency Subcommittee on Sedimentation, and dedicated to the improvement and maintenance of the rivers and harbors of the nation, awaits with considerable interest the research and findings of the agencies represented in that group.

DISCUSSION

D. J. HEBERT.* In this paper, Messrs. Simmons, Fenwick, and Patterson have clearly and forcefully demonstrated that the hydraulic model is a powerful tool for use in studying sedimentation problems in navigation channels. The three examples given are well-chosen to illustrate the flexibility of laboratory methods of study.

The three model investigations described in the paper are treated so completely that there remains very little to be added by way of comments on testing procedure and results. It might be worthwhile to point out that the process of verification, which appears to be relatively simple as it is described by the authors, is really quite a difficult and involved procedure. Being faced with the necessity for making adjustments in the various model parameters the best choice ulti-

* Hydraulic Laboratory, Branch of Design and Construction, Bureau of Reclamation, Denver, Colo.

mately depends upon the operator's judgment. This in turn derives from his understanding of the hydraulic action involved and from his experience. The verification must be extended to cover a range which includes as many different conditions as possible. This will tend to minimize the possibility that a new set of conditions might not be covered by the similitude relationships developed in the verification procedure.

In studies involving considerations of tidal prism such as the Savannah Harbor investigation, it should be remembered that tidal prism is the volume actually filled by water between instantaneous water surface profiles at high and low stage. It should not be confused with tidal volume, which is a gross volume between level water surfaces at the two stages. Whether or not changes in tidal volume will result in changes in tidal prism is not always apparent. For example, let us take the case of the Zuider Zee Dam in Holland as described by Van Veen in an unpublished paper. The dam would extend across a bay and completely eliminate a large portion of tidal volume. This raised the question of the possible decrease in tidal currents past the Friesian Islands which are located seaward from the dam. Studies, later confirmed by observations after the dam was completed, showed that actually the currents were increased by some 20 percent. The actual change in tidal prism which is closely related to the tidal currents is not known but it is certain that it was not proportional to the change in tidal volume.

I would like to underscore the statement that model-prototype confirmation tests are invaluable and should be considered as part of the study. In some instances where the data are meager, the prototype tests are mandatory. The main function of the model in such cases is to clarify the basic principles involved and to develop the techniques whereby determination of critical constants may be made on a prototype level. In the case of a model study of tidal intrusion of ocean salinity into the channels of the Sacramento-San Joaquin Delta, the model was used principally to demonstrate that the intrusion was a true diffusion process accomplished by mixing during tidal ebb and flood. The coefficients of diffusion at various critical locations in the delta can be determined accurately only by measurement in the prototype.

The men who have been working in the larger hydraulic laboratories and have contributed to the development of the hydraulic model as a design tool have, in the past, been unable to devote as much time as they should to the fundamental aspects of problems. One of the major factors responsible for this situation is the necessity for meeting schedules. This is a mistake because every model study, especially when a sediment problem is involved, can and should be made to yield some contribution to the fund of basic knowledge. The impetus required to correct the situation will have to come from the laboratory staffs in the form of comprehensive research programs.

The laboratory men are in a position to aid materially in formulating the overall program called for by Dr. Mason in a previous discussion. They have one root in theory and one in practice particularly when confirmation studies or extensions of model studies to prototype-level are involved. They should be able to do much to supply the link between basic and practical aspects of any program.

ROBERT T. KNAPP.* The applicability of model techniques to the study of sedimentation problems has been an open question for a long time. The present paper by Simmons, Fenwick and Patterson gives a very clear-cut description of three different methods of attack that are now being employed at the Vicksburg Laboratory. These methods might be defined as follows:

A. The movable bed model which attempts to simulate the entire sediment movement in a quantitative manner.

B. The fixed bed model in which shoaling or deposition is investigated through the injection of finely-ground material into the flow.

C. The fixed bed model without sediment in which the investigation is confined to the study of the hydraulic forces which influence erosion and deposition.

Before any attempt is made to comment on the applicability of these different model techniques, I should like to consider some of the broader aspects of hydraulic laboratory research.

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* Cooperative Agent, Soil Conservation Service, California Institute of Technology, Pasadena, Calif. (Presented by V. A. Vanoni).

It is possible to divide laboratory techniques for studying hydraulic problems into a number of different classes, as shown in Table 1. Of course, these classes are not mutually exclusive, there is much overlapping and combining. Nevertheless, the classification is useful if it serves to emphasize any basic differences that may exist.

1. In the first class may be placed studies of the fundamental nature and mechanism of a specific hydraulic phenomenon. These are usually paralleled by analytical studies since the combination of the analytical and experimental method often makes much faster progress than either one alone. The objective of this class of studies is to achieve so thorough an understanding of the phenomenon that it may be treated analytically and thereby eliminate the necessity for further experimental investigation.

2. This and the subsequent classes may all be considered model studies. To fall into this class, a study should fulfill the following conditions:

   a. Basic characteristics of the phenomenon involved should be well understood.

   b. The need for a model study should arise from the fact that the application is too complicated or involved to be handled readily by analytical means.

   c. It should be possible to make the study within the bounds of strict model similarity.

3. This class contains model studies having the following characteristics:

   a. Basic characteristics of the phenomenon involved should be well understood.

   b. The need for a model study should arise from the fact that the application is too complicated or involved to be handled readily by analytical means.

TABLE 1. - CLASSIFICATION OF LABORATORY STUDIES

Study Characteristics	Classification				
	1	2	3	4	5
Basic laws governing phenomenon	Known	Known	Known	Known	Unknown
Type of solution sought	General	Specific (model study)	Specific (model study)	Specific (model study)	Specific (model study)
Number of major Phenomena involved	1	1	1	2 or more	1 or more
Degree of model similarity obtainable	Not involved	Strict	Small deviation	Major deviation	Unknown
Need for verification from prototype	Not required	Not required	Desirable	Required	Imperative
Estimated reliability	Excellent	Excellent	Satisfactory	Uncertain	Unknown

   c. It is inconvenient or impractical to construct a model having strict geometric and dynamic similarity. The departures from similarity, however, should be deliberate and based upon a knowledge that such departures should produce effects whose magnitudes are of second order.

4. Studies in this class are characterized by conditions which involve the interaction of two or more phenomena which cannot all be studied in one model without substantial deviations from strict similarity.

5. This class contains studies involving unknown phenomena for which it is impossible to evaluate conditions required for similarity.

In general, the engineer is not free to choose the class of study which must be carried out. The nature of the problem determines that for him. Furthermore, the information obtained even from class 5 studies is often extremely valuable and worth many times the cost of obtaining it. Therefore, none of the remarks that follow should be considered as a criticism of the work of any sound laboratory, and certainly not of the organization to which the authors of the paper belong. They merely represent the writer's philosophy concerning model studies and their applicability to the solution of sedimentation problems.

It is felt that these classifications of laboratory studies have been arranged in order of decreasing reliability. If this classification of laboratory studies is accepted, it will be seen that models used for studying sedimentation problems nearly always fall into classes 4 and 5. The reliability of their results depends in a large measure upon such factors as the amount of field information available, the knowledge and skill of the experimenter, and chance. Chance probably plays its largest role in studies coming under class 5.

Classes 1 and 2 should require no verification to establish their validity.

Class 3 studies should require very simple checks or verifications, whereas the reliability of class 4 and 5 studies is based almost entirely upon the verification of model performance with measured performance in the field. One of the difficulties of such verification is that it is seldom possible to carry it out for more than one condition. It would be much more comfortable to be able to have at least two verifications, one at each end of the range of conditions involved in the experiments. After all, the model is a measuring instrument and every experimenter has much more confidence in an instrument that he has been able to calibrate over its extremes of range than on one for which he has only a single calibration point.

It is the belief of the writer that in enumerating three types (a, b and c) of model techniques for studying sedimentation problems that they have been presented in the order of the greatest apparent reliability, but there is at least a suspicion that the true reliability may be in the reverse order. Certainly the movable bed model, in which the entire cycle of cutting and deposition takes place, is by far the most convincing to the non-specialist even though he may be an engineer. The operation of the fixed bed model with injected shoaling material is apt to be not quite so convincing, but it still gives a pretty good visual picture as to what is happening. The fixed bed model which uses no sediment at all is apt to appear as an abstraction since the behavior of the sediment cannot be seen visually but must be predicted from a series of curves and tables. The other side of the picture is, however, that the movable bed model involves the greatest unknowns and in general the most serious violations of the similarity conditions. It is felt that in many cases its convincing appearance is one of its greatest drawbacks since the observer and even the investigators themselves are apt to place undue reliance upon the results.

On the other hand, the fixed bed model without sediment injection can usually be constructed with only minor violations of the similarity laws. The big advantage is that no attempt is made to simulate the actual sediment movement. When the investigator has to infer the nature, location, and magnitude of the sediment movement from the velocity distributions measured in the model and the laws now available concerning sediment transportation, it is difficult for him to avoid being aware of the large uncertainties involved.

A few paragraphs ago reference was made to the role played by chance in class 4 and 5 studies. In their paper, the authors present an example of its effect. In discussing the study of the Deep Water Point Range, a comparison was given between the model results of the effect of the construction of the Pennsville Dike and the prototype results measured after the dike was actually built. The model showed a reduction in shoaling of 47 percent; whereas, the prototype measurements indicated a reduction of 48 percent. This extremely close agreement would seem to be quite fortuitous. If either the model results or the prototype measurements were accurate within 10 percent they should be considered to be very good.

The writer agrees wholeheartedly with the authors in their concluding remarks concerning the importance of carefully planned model to prototype confirmation studies for sedimentation models. As the paper points out, the sedimentation model can be made to provide much valuable information by other means. Some of this information is inherently empirical in nature, and is applicable only to the specific conditions of the study. However, it is believed that unless sedimentation models are also employed to extend the fundamental knowledge of the mechanics and laws of sedimentation itself, they will be failing to capitalize upon the possibilities inherent in them, and thus not doing their part to increase the reliability of this important class of model studies.

LABORATORY EXPERIENCES WITH THE BOTTOM WITHDRAWAL TUBE METHOD OF SIZE ANALYSIS.

by C. S. Howard. *

The bottom withdrawal tube method 1/ of size analysis has been used in the Southwestern Laboratory of the Geological Survey for nearly three years. During that time over one thousand samples have been analyzed by this method. In the early part of the work the settling times recommended in Report No. 7 1/ were followed. Laboratory tests indicated, however, that neither of the groups of settling times (10 seconds to 120 minutes recommended for coarser particles, and 4 minutes to 124 minutes recommended for finer particles) gave complete records for the sediments being analyzed in this laboratory. Neither group of settling times gave sufficient data for computing the percentages of particles finer than 0.0039 mm., and the times recommended for the finer particles gave too few data for satisfactory computations of the percentages of particles with diameters greater than 0.0625 mm. Many samples analyzed in this laboratory have considerable percentages of the finer particles, other samples collected from the same stream within a short period of time have considerable percentages of the coarser particles, and it is difficult to select a satisfactory set of settling times by inspection of the sample. The range in the particle sizes in samples from the same river is illustrated by the graphs in Figure 1.

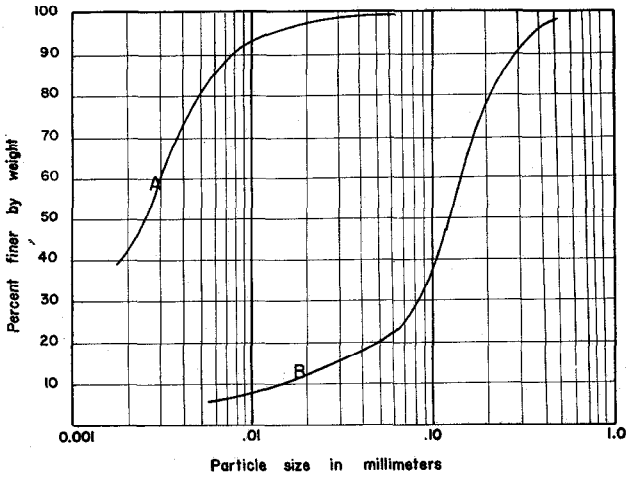

Figure 1. Size analyses of suspended sediment in samples from the Colorado River near Grand Canyon, Ariz.

A, April 3, 1945 B, May 30, 1945

In this laboratory an attempt was made to have the data sufficiently complete so that less than twenty five percent of the material was coarser than the coarsest size reported and less than twenty five percent was finer than the finest size reported. A large number of size groups have been chosen between the coarsest and finest sizes but no attempt was made to select intermediate size groups within this range, even if the percentage of material in any of the reported sizes was greater than twenty five percent. To obtain adequate data for the coarser particles, the total settling time was short, and a number of withdrawals were made in the first few minutes of settling. For many samples, a total settling time of ten minutes was chosen and four withdrawals made in the first 1 3/4 minutes. For a few of the samples more than ninety five percent had settled in ten minutes but for most samples a longer settling time was needed. Total settling times of twenty and forty minutes were used on an appreciable number of samples and for many of these samples the results were adequate for drawing the Oden curves. Many of the samples had large percentages of the finer particles, and a total settling time of forty minutes did not give sufficient data for determining the distribution of the sizes of the finer particles. For those sampled for which it was assumed that the sediment had large percentages of the finer particles, a total settling time of 130 minutes was selected and in later determinations the total settling time was 400 minutes. In some of the analyses made with 400 minutes as the total time the data were not sufficient to determine accurately the percentages of the particles with diameters greater than 0.0625 mm., and there could be no break-down for the particles with diameters less than 0.00195 mm. Many of the samples had more than forty percent of the particles with diameters less than 0.00195, but a total settling time much in excess of 400 minutes is not feasible.

* District Chemist, Geological Survey, Albuquerque, N. Mex.

1/ Iowa Inst. of Hydraulic Research, "A Study of New Methods for Size Analysis of Suspended Sediment Samples." Report No. 7 of "A Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams," 1943.

For samples which had considerable percentages of the coarser particles, i.e., when the material in the first three or four withdrawals amount to more than twenty five percent of the total material, the material drawn off in the first three or four withdrawals was sieved. The number of withdrawals chosen for the sieve analysis was selected by consideration of the settling velocities and choosing those portions which were withdrawn during the time when all of the particles having diameters greater than 0.0625 mm. would have fallen through the entire 100 cm. The sieves used had openings of 0.500 mm., 0.250 mm., 0.125 mm., and 0.0625 mm. The results of the sieve analyses were often in poor agreement with the results of the bottom withdrawal tube analyses for the same samples. At first, it was thought that the differences were because the particles were not spherical and had a specific gravity different from the assumed value of 2.65 used in computing the settling times. Further study indicated that the discrepancies were probably due, in large part, to the few points available for plotting the percentages of coarse material. Comparable results for twelve samples from the Colorado River at Grand Canyon and forty nine samples from the San Juan River near Bluff, Utah, are given in Table 1.

TABLE 1. COMPARATIVE RESULTS FOR ANALYSES OF SAMPLES OF SUSPENDED SEDIMENT ANALYZED BY THE BOTTOM WITHDRAWAL TUBE METHOD (A) AND BY SIEVING (B).

Location	Method of Analysis	Percent finer than indicated size(in millimeters)			
		0.0625	0.125	0.250	0.500
Grand Canyon (12 samples)	A	40	58	81	91
	B	42	64	91	99
Bluff (49 samples)	A	50	60	79	91
	B	47	60	87	98

The study of the bottom withdrawal tube method in the Southwestern Laboratory of the Geological Survey was coordinated with the studies made in the Albuquerque laboratory of the U.S. Engineers. The Engineers adopted a schedule of settling times (I, Table 2) with a total settling time of 400 minutes. Their schedule provides for four withdrawals within the first two and a half minutes, that is during the period when the coarser particles are settling. A modified form of the Engineers schedule has been used by the Geological Survey (J, Table 2).

The methods of plotting the analytical data outlined in Report No. 7 were modified for use in this laboratory. The Oden curves made in this laboratory were made on graph paper having 20 by 20 lines to the inch. Percentages were plotted with 100 percent at the base line, and with a percentage scale of one inch representing 20 percent. In the early work, one inch on the time scale for the finer particles (curve A) represented 200 minutes. Later this scale was changed to have one inch represent 400 minutes. For many of the samples it was found desirable to have four time scales, and for a number of samples five time scales were useful. In the more recent work in this laboratory the following scales were used: Curve E, one inch representing 800 minutes; Curve A, one inch representing 400 minutes; Curve B, one inch representing 40 minutes; Curve C, one inch representing 4 minutes; Curve D, one inch representing 0.4 minutes.

The laboratory work sheets consist of (1) bottom withdrawal tube size analysis sheet (Figure 2), used for recording the weights of the various withdrawn portions and for computing the percent in suspension after different settling times, and (2) graph sheet for drawing the Oden curve and computing the size analysis (Figure 3). A copy of each sheet is attached. The Oden curve sheet shows the size groups for which percentages were computed. Very few of the samples analyzed in this laboratory had any appreciable quantities of material with diameters larger than 0.500 mm. The analytical results are tabulated on a sheet similar to that used for table 5. It will be noted that some of the sizes reported on the Oden sheet are not given in this table, the condensation was made to keep the tables from being too bulky.

The withdrawal times used by the Geological Survey are shown in Table 2. Percentages and withdrawal times for twelve samples are shown in Table 3. For some analyses two or more time scales were used in drawing the Oden curves, and size analyses for nine of these samples are given in table 4. Two sets or curves were drawn for each of these analyses and three sets were drawn for one analysis. The agreement between the different analyses was usually within two percent and it is likely that this agreement may represent the reproducibility that can be expected

Table 2. Settling times for bottom withdrawal tube method of size analysis.

A		B		C		D	
Elapsed time	Time to settle 100 cm. (min.)	Elapsed time	Time to settle 100 cm. (min.)	Elapsed time	Time to settle 100 cm. (min.)	Elapsed time	Time to settle 100 cm. (min.)
15"	0.28	27"	0.50	27"	0.50	27"	0.50
27"	.56	48"	1	48"	1	48"	1
48"	1.14	1'24"	2	1'24"	2	1'24"	2
1'45"	2.92	3'	5	3'	5	3'	5
2'24"	4.8	7'30"	15	7'30"	15	10'	20
3'	7.5	10'	25	20'	50	16'	40
4'48"	16	12'	40	27'	90	27'	90
7'30"	37.5	16'	80	30'	150	35'	175
10'	100	20'	200	40'	400	40'	400
10'		20'		40'		40'	
E		**F**		**G**		**H**	
54"	1	54"	1	15"	0.28	4'30"	5
4'	5	4'	5	50"	1.04	10'	12.5
10'	14	10'	14	1'24"	2	17'30"	25
24'	40	24'	40	3'	5	30'	50
50'	100	50'	100	5'	10	50'	100
80'	200	70'	175	16'	40	100'	250
90'	300	90'	300	30'	100	250'	833
100'	500	100'	550	80'	400	300'	1500
120'	1200	130'	1300	120'	1200	400'	4000
120'		130'		120'		400'	
		I		**J**			
		9"	0.17	54"	1		
		15"	.31	4'	5		
		42"	1.0	9'	13		
		2'30"	4.2	18'	30		
		7'30"	15	40'	80		
		25'	62.5	60'	150		
		75'	250	150'	500		
		200'	1000	240'	1200		
		400'	4000	400'	4000		
		400'		400'			

for analyses made by the bottom withdrawal tube method.

Few of the samples received in this laboratory had sufficient water for the bottom withdrawal tube method. An effort was made to use for the settling water a water with a concentration and character of dissolved solids similar to the concentration and character of the water in which the suspension was carried. For many of the samples it was possible to use in the analysis water from the same gaging station or from another one in the same river basin, but for other samples it was necessary to use Albuquerque tap water. A number of samples were dispersed before analysis, using sodium oxalate as the dispersing agent and distilled water as the settling medium.

Many of the samples received in this laboratory had concentrations of sediment much higher than the maximum of 10,000 parts per million recommended in Report No. 7. It was necessary to split those samples and for that purpose a splitter was used which was like the one developed by the Albuquerque laboratory of the U.S. Engineers. This splitter is similar to the Jones ore splitter and worked very well. Analyses were made of several portions of a number of samples.

Size analyses for portions of some of the split samples showed good agreement but for most of the comparable samples the agreement was poor. Analytical results for a few of the samples are given in table 5. As a rule the analyses made with the higher concentrations of sediment

BOTTOM WITHDRAWAL TUBE SIZE ANALYSIS

River _____ Type sample _____

Station _____ Sediment concentration _____ percent

Date _____ _____ ppm

Elaps. time min.	Fall ht. cm.	Dish No.	Weight: Gross (gm.)	Tare (gm.)	Sediment (4)-(5) (gm.)	Net (6)-Dis. solids (gm.)	Cumulative (gm.) Read up	Depth factor 100/(2)	Sed. in susp. 100 cm. Fall (8)x(9) (gm.)	Percent in susp.	Time to settle 100 cm. (1)x(9) (min.)
1	2	3	4	5	6	7	8	9	10	11	12

Lab. No. Analyzed by Date Tube No. Temp. °C. Checked by Date

Suspended sediment
Sample bottle +
Sample bottle No. _____
Suspension _____ gm.

Dissolved solids
_____ ml. _____ gm/ml.
Dish +
Dish No. _____ gm.

FIGURE 2. Sample of bottom withdrawal tube size analysis sheet.

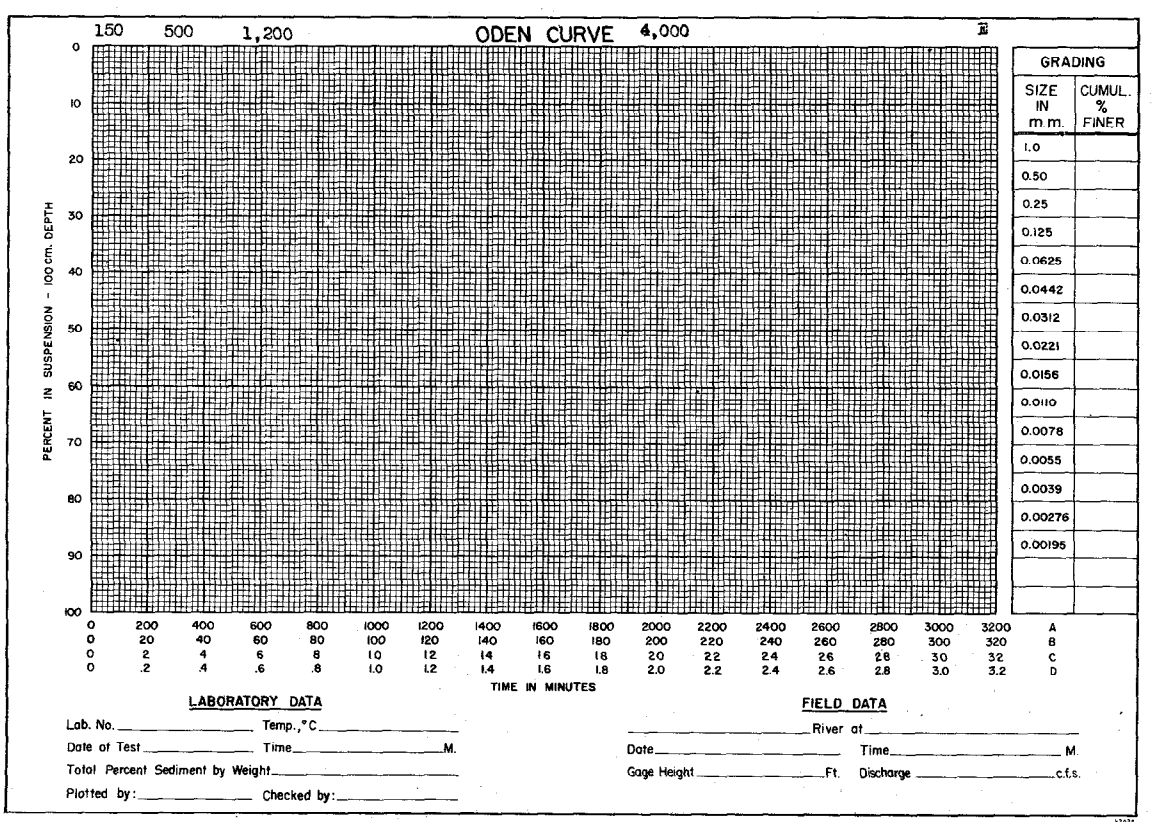

FIGURE 3. Sample of plotting paper for Oden curve

Table 3. Settling times and percentages for twelve sediment samples.

	Settling schedule A			Settling schedule B			Settling schedule D		
	Time to settle 100 cm. (minf)	1 Percent in suspension	2 Percent in suspension	Time to settle 100 cm. (min.)	1 Percent in suspension	2 Percent in suspension	Time to settle 100 cm. (min)	1 Percent in suspension	2 Percent in suspension
	0.28	91	91	0.5	62	75	0.5	79	43
	.56	79	86	1.0	40	57	1.0	72	32
	1.14	62	77	2.0	26	42	2.0	69	24
	2.92	38	65	5	20	31	5	66	17
	4.81	28	56	15	15	24	20	62	12
	7.5	22	48	25	13	22	40	57	10
	16	17	35	40	12	20	90	42	9
	37.4	15	30	80	11	18	175	25	8
	100	14	21	200	10	16	400	16	6
Total sediment (gms)		3.2140	4.7520		1.7771	1.1606		4.3753	1.9394
Total settling time (min)		10	10		20	20		40	40
Temp. °C.		25	23		31	28		27	28
Adequacy of Analysis		good	fair		good	fair		good	fair

	Settling schedule F				Settling schedule J			
	Time to settle 100 cm. (min)	Percent in suspension	Percent in suspension	Percent in suspension	Time to settle 100 cm. (min.)	Percent in suspension	Percent in suspension	Percent in suspension
	1	86	85	99	1	92	97	66
	5	74	71	95	5	83	90	59
	14	68	63	89	13	77	85	56
	40	43	55	78	30	68	79	53
	100	6	47	69	80	57	70	48
	175	3	41	63	150	50	64	42
	300	2	35	57	500	33	53	18
	550	1	30	51	1200	21	47	14
	1300	0	26	43	4000	10	39	8
Total sediment (gms)		2.6810	1.0826	4.6120		0.5530	0.6842	1.1374
Total settling time (min)		130	130	130		400	400	400
Temp. ° C.		26	23	20		26	27	25
Adequacy of analysis		good	fair	fair		good	fair	fair.

Description of samples.

A-1 Alamogordo reservoir deposit, dispersed with sodium oxalate
A-2 Conchas reservoir deposit, dispersed with sodium oxalate

B-1 San Juan River near Bluff,Utah, June 18,1944
B-2 San Juan River near Bluff,Utah, June 19, 1944

D-1 San Juan River near Bluff, Utah June 4, 1944
D-2 San Juan River near Bluff, Utah, July 21, 1944

F-1 Arkansas River near Las Animas, Colo., July 25, 1944
F-2 San Juan River near Ship Rock, N. Mex., April 15, 1944
F-3 Alamogordo reservoir deposit, dispersed with sodium oxalate

J-1 Rio Grande at Albuquerque, N. Mex., Aug. 17, 1945
J-2 Rio Grande at Albuquerque, N. Mex. Aug. 17, 1945
J-3 San Juan River near Bluff,Utah, Mar. 25,1945.

Table 4. Size analyses for nine sediment samples.

(Lines numbered a, b, and c represent first, second, and third plotting, respectively)

No.	Time scale (min. per inch)	Percent finer than indicated size in millimeters									
		0.0039	0.0055	0.0110	0.0156	0.0312	0.442	0.0625	0.0125	0.250	0.500
A-1-a	200	-	-	-	16	21	30	48	84	98	-
A-1-b	400	-	-	14	15	20	30	48	83	99	-
A-2-a	200	-	-	-	32	47	62	73	90	95	-
A-2-b	400	-	-	32	34	45	63	76	89	96	99
B-1-a	400	-	-	13	15	20	22	31	57	92	98
B-B-b	400	-	-	11	13	19	22	30	59	94	99
B-2-a	400	-	-	19	21	29	34	46	75	97	99
B-2-b	400	-	-	-	22	29	35	43	74	97	100
D-1-a	400	-	-	10	11	15	19	26	41	62	81
D-1-b	400	-	-	10	11	16	18	23	41	61	79
D-2-a	400	-	18	52	65	67	68	70	76	91	97
D-2-b	400	-	16	55	65	67	68	70	76	90	97
F-1-a	400	-	2	9	65	79	81	83	90	95	98
F-1-b	400	-	2	9	67	74	79	83	91	94	96
F-2-a	200	29	37	56	61	68	71	81	90	96	-
F-2-b	400	29	37	54	59	67	74	82	90	94	96
F-2-c	100	-	38	54	59	69	74	80	92	97	99
F-3-a	200	48	58	74	85	95	98	98	-	-	-
F-3-b	400	53	60	74	81	94	97	99	100	-	-

Sample numbers, A-1, A-2, B-1, B-2, D-1, D-2, F-1, F-2, and F-3 refer to sample numbers in Table 3.

showed smaller percentages of the finer particles than was obtained with lower concentration. This was especially true when the material consisted chiefly of the finer particles (diameters less than 0.0055mm.) and when the settling medium was native water. The material, as it settled in the more concentrated suspensions, appeared to settle like a coagulated mass.

On several occasions it has been observed that a moderately or highly concentrated suspension settles more completely and faster than a less concentrated suspension of the same material, and also that the settlement in a narrow cylinder is more complete than in a wider cylinder. Observations made in the Southwestern Laboratory on the rate of settling of a sample of Rio Grande water in four cylinders showed that for concentrations greater than 6,000 parts per million the settling was more complete in the smaller cylinders. The cylinders used had the following diameters: 6.9 cm., 4.6 cm., 3.6 cm. and 2.6 cm. The studies were not sufficiently complete to show the effect of the diameter of the settling cylinder on the rate of settling of different suspensions.

The available analytical data show that comparable results cannot be obtained for certain samples if the concentrations of sediment in the settling tubes are different and if the settling is done in native water. It appears that the concentration, if less than 5,000 parts per million, is not a factor in getting comparable results when the sample is dispersed and settled in distilled water. Analyses representing three settling conditions for a sample from the Rio Grande are shown in Figure 4. Each curve is the mean of several analyses made on split portions having similar concentrations of sediment. The curves for this one sample and the data obtained on a large number of samples show the improbability of getting comparable results for different concentrations of sediment when the material is fine and the settling is in native or tap water. Comparable results can be obtained for different concentrations of sediment when the sample is dispersed and the settling done in distilled water, but it is not known how high concentrations of sediment should be used with dispersed samples.

Table 5. Size analyses of suspended sediment from various sources.

(Analyses by the bottom withdrawal tube method, using the "J" schedule for settling times)

Date of collection	Concentration of sediment in tube (ppm)	Percent finer than indicated size (in millimeters)											Settling medium
		0.00195	0.00276	0.0039	0.0055	0.0078	0.0156	0.312	0.0625	0.125	0.250	0.500	
				Rio Puerco at Adamana, Arizona									
May 2, 1945	521	29	55	77	85	88	90	93	96	99	100	-	Tap water
	624	50	62	76	85	91	96	100	-	-	-	-	Tap water
	1,220	13	35	54	78	88	94	96	98	99	100	-	Tap water
	1,220	15	45	62	85	92	94	96	99	100	-	-	Tap water
	2,330	9	15	27	44	83	93	96	98	100	-	-	Tap water
	2,400	6	10	30	44	95	98	100	-	-	-	-	Tap water
				Rio Grande at Albuquerque, N. Mex.									
Aug. 17, 1945	341	42	47	51	58	75	85	93	98	99	100	-	Native water
	1,050	14	25	36	47	57	71	83	92	95	99	100	Native water
	2,280	10	14	19	28	50	77	91	98	99	100	-	Native water
	2,560	47	54	61	69	75	85	93	97	98	100	-	Distilled water (dispersed)
				Rio Grande at San Acacia, N. Mex.									
July 23, 1944	1,700	25	28	61	76	81	88	95	99	100	-	-	Tap water
	6,100	-	-	30	45	68	83	94	-	-	-	-	Tap water
July 17, 1946	194	43	50	56	64	82	96	999	-	-	-	-	Native water
	820	-	4	8	50	70	95	99	-	-	-	-	Native water
	2,050	12	18	28	49	70	95	99	-	-	-	-	Native water
	2,150	56	62	70	79	89	97	100	-	-	-	-	Distilled water (dispersed)
				Grand River near Shadehill, S. Dakota									
March 23, 1946	246	-	-	-	56	-	-	-	-	-	-	-	Native water
	392	-	36	42	46	90	96	100	-	-	-	-	Native water
March 30, 1946	291	-	4	21	37	48	-	-	-	-	-	-	Native water
	352	-	-	34	36	50	72	88	94	100	-	-	Tap water
	219	-	94	-	-	-	-	-	-	-	-	-	Distilled water (dispersed)
	551	-	94	-	-	-	-	-	-	-	-	-	Distilled water (dispersed)
				Colorado River near Grand Canyon, Arizona									
Oct. 7, 1944	955	38	48	55	82	95	99	-	-	-	-	-	Tap water
	4,090	-	7	10	26	61	99	-	-	-	-	-	Tap water
				San Juan River near Bluff, Utah									
Mar. 28, 1945	917	18	32	49	64	69	71	72	76	80	94	97	Tap water
	2,080	9	15	17	34	55	-	-	1/ 56	1/ 66	1/ 93	1/ 99	Tap water
Oct. 17, 1945	3,790	-	-	2	5	6	55	-	1/ 61	1/ 72	1/ 94	1/100	Native water
	4,180	35	39	44	53	57	62	65	1/ 67	1/ 74	1/ 98	1/100	Distilled water (dispersed)
Apr. 26, 1946	4,880	-	-	8	16	23	35	54	1/ 64	1/ 74	1/ 93	1/100	Native water
	7,350	-	15	22	26	28	38	53	1/ 66	1/ 75	1/ 93	1/100	Distilled water (dispersed)

1/ Determined by sieve method.

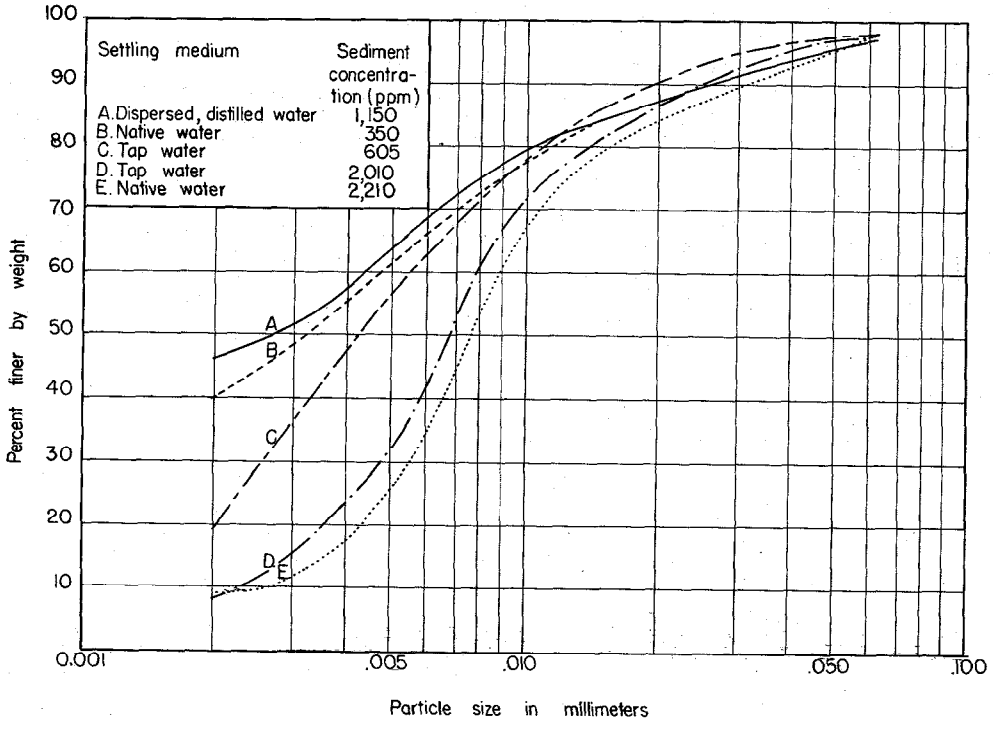

Fig. 4. Bottom withdrawal tube size analyses of Rio Grande sediment for different concentrations in different settling media.

The available information indicates that for samples dispersed with sodium oxalate it may be difficult to ascertain the correction that should be applied for the dissolved solids of the settling medium. The weight of dissolved solids on several samples appears to be less than the weight computed for the amount of sodium oxalate added, indicating a reaction between the oxalate and the suspension or solution. The necessity for a correction for the dissolved solids may be eliminated by filtering through a Gooch crucible.

During the settling, most of the samples showed an appreciable amount of sediment collecting on the shoulder at the bottom of the withdrawal tube. A few observations show that some material is deposited on the shoulder during the first few minutes of settling but the rate of deposition is not known for the different withdrawal periods of the analysis. In this laboratory the material that collects on the shoulder during the entire analysis is washed out with distilled water and weighed. The weight is then added to the weight representing the material that settled during the ninety minute period between the 60 and 150 minute withdrawals. The records obtained in this laboratory indicate that the amount of material which collects on the shoulder of the tube may amount to four percent of the total quantity of material in the tube.

On the basis of the analytical data obtained in the Southwestern Laboratory the following procedure is recommended for the analysis of sediments by the bottom withdrawal tube method:

1. The native water will be removed from the sample. If necessary the sample will be split to get a portion which give a concentration of sediment in the tube less than 5,000 parts per million. Most of the samples will be dispersed before analysis but two or three samples each month will be settled in native water using one or more concentrations of sediment (these concentrations to be less than 2,500 parts per million). For dispersion add about 100 ml. of distilled water, 5 ml. of sodium oxalate solution (1.5 °/o) and mix the suspension well by stirring with a milk shake mixer.

2. Most of the samples will be analyzed using the following settling times: 54",4',9', 18',40',60',150',240', 400' and 400'. If most of the material settles out within the first 60 or 150 minutes (as indicated by a light suspension left in the tube) the remainder of the suspension can be drawn off at five-minute intervals, except the last portion is to be drawn off immediately after the next to the last portion.

3. The temperature of the portion drawn off at the end of 150' is recorded as the

temperature of the entire suspension during the analysis.

4. If the material that settles in the first nine minutes (the material in the first three dishes) amount to more than twenty five percent of the total amount of material in the tube, a sieve analysis should be made of the material in the first three dishes, using sieves with openings of 0.500,0.250,0.125,and 0.0625 mm. The results of the sieve analysis are to be reported in the size analysis.

5. After the last portion has been drawn off, the tube should be washed out with a stream of distilled water to remove any material clinging to the sides of the tubes (especially on the shoulder at the bottom). The material washed out is to be weighed and the weight added to the weight of the material that settled during the settling period from 60' to 150'.

6. In the absence of any determined values, the correction for dissolved solids on the dispersed samples may be assumed to be 0.0050 grams for each withdrawal. The correction for dissolved solids in the samples settled in native water will be determined by evaporating a 50 ml. sample of the native water, drying and weighing the residue.

DISCUSSION

MARTIN E. NELSON. * The maximum velocity attained by a body in a free fall in a gaseous or liquid medium is a function of the physical dimensions of the body, its density, and the density of the surrounding fluid. The mathematical expression for the terminal velocity was first published by Stokes in 1845. About 40 years later engineers and geologists began to apply this law in determining particle size gradations of sand, sediment, and other granular materials. Although the terminal velocity of sand and sediment particles is affected by the body form, it can be related to the diameter of spherical particles with equivalent sedimentation characteristics. In the majority of sediment problems it is not the exact geometric size or shape of the individual particles that is of primary importance but rather their behavior in regard to transportation and deposition in rivers and reservoirs, which factors are directly related to the sedimentation characteristics of the particles. Therefore, it is obviously advantageous to make size analyses of fluvial sediments by a method which is based on these characteristics.

During the past fifty years several ingenious procedures have been developed for the purpose of determining particle sizes by sedimentation methods, such as decantation, elutriation, manometer, hydrometer, plummet, and pipette. The limited accuracy and range of application, as well as the tedious processes involved in applying these methods, have provoked further search for simpler and more accurate methods of making sediment size analyses. These circumstances prompted the Federal agencies who have cooperated in the investigation of sediment measurement and analysis at Iowa City to sponsor the development of the bottom-withdrawal tube.

The greater sensitivity of the bottom-withdrawal tube analysis is attended by new problems which have not concerned the laboratory technician using less accurate methods. As Mr. Howard has indicated in his very instructive review of experiences with the bottom-withdrawal tube, the problem of flocculation assumes new and greater importance. Undoubtedly, flocculation plays an important role in the natural sedimentation processes occurring in rivers and reservoirs. This is true particularly when finely divided sediment particles are present. The laboratory technician is confronted with the problem of simulating conditions existing in nature, in the apparatus he uses in his sedimentation analysis. Unfortunately we do not know just what forces are involved in a natural sedimentation process. It is not conclusive to say that as the stream velocity is reduced to a certain value all particles of a given size will settle out and come to rest on the bottom. The process may be accelerated by the presence of certain ions of dissolved solids and it may be retarded by others. The mixing of different waters as one river flows into another or into a reservoir may create circumstances which cause fine suspended sediment to react differently than would similar sediment in other waters.

In order to facilitate the interpretation of the sediment data, the size analysis should be presented in a form that enables the engineer to determine its effect upon river regulation or development projects. Usually this effect is related to the settling characteristics of the sediment particles. The size analysis determined by a sedimentation process in a completely dispersed suspension will indicate the gradation of the true diameters of the suspended material. When this method of size analysis is used on a sample in which flocculation occurs, the analysis

* Corps of Engineers, Department of the Army, St. Paul District, St. Paul, Minn.

will indicate a size gradation coarser than the actual. While it is true that flocculation does take place in the natural stream, there are no criteria available at the present time for determining the degree of flocculation that would occur under the countless variations possible in the contributing factors. The laboratory technician is unable to induce dispersion in the sample with any assurance that the settling characteristics of the sediment particles in the stream from which the sample was taken will be simulated.

There appears to be a new field of research which should be covered in order to clarify this phase of sedimentation. In the meantime, it is my opinion that the effects of dissolved solids should be eliminated as far as possible when making laboratory size analyses with the bottom-withdrawal tube. Sediment data determined from size analyses in which no flocculation occurred would be more readily interpreted than data obtained from analyses in which an uncertain degree of flocculation took place. Therefore, until the effects of flocculation in the laboratory apparatus and that which occurs in nature can be correlated, the bottom-withdrawal tube size analyses should be made in distilled water with the assistance of a dispersing agent when necessary.

The bottom-withdrawal tube in its present state of development is not necessarily the ideal instrument for making size analyses of sediment samples. Improvements in the instrument and auxiliary equipment will no doubt be developed with its further use. The technique of operating the tube and analyzing the data should be facilitated so as to reduce the cost and labor involved. However, sufficient tests have already been made with this instrument in various laboratories to demonstrate that it is practical, that it is superior to other methods for accuracy, and that it is applicable to wider ranges of concentration and particle size gradation than other methods commonly used in making sediment size analyses.

E. W. LANE.* Dr. Howard has very forcefully brought out the difficulties of sediment size analysis, particularly those introduced by different concentrations in the sediment analyzed and differences in the chemical composition of the water. Although he has discussed this as an aspect of the bottom withdrawal tube, it is really a broader problem, for it applies equally well to the analysis by any device using the rate of settlement in water, such as the hydrometer or pipette. He has shown that widely different results may be obtained with the same material under different conditions, and the question, therefore, arises as to which condition we should adopt as our standard in order that the results obtained throughout the sediment field would be comparable. To decide this question, it is necessary to consider the use which will be made of this sediment data, and to select the conditions of analysis that will give data in the form most suitable for that use.

Unfortunately, this approach indicates that more than one method of analysis should be used. For example, suppose that it is desired to determine the amount of sediment that will be deposited as a flood passes through a retarding basin, which is the problem discussed by the writer on another page. For this purpose, it is necessary to know how fast the particles will settle in the water in which they flow and at the concentration which exists there. For this purpose, the samples would have to be analyzed under as nearly as possible the conditions which existed where and when the sample was taken. However, suppose it is desired to determine the size of the sediment particles which have passed through a reservoir without depositing by comparing the composition of the material deposited in the reservoir with that which flowed in, as shown by the samples taken from the reservoir bottom and from the inflowing stream. For this purpose, the analysis of material both from the reservoir deposits and from the inflow would have to show the ultimate particle size, and both analyses would have to be made in a highly dispersed condition in order that the particle settling rate is not affected by the proximity of adjacent particles. The analysis made in the manner best suited to determine the deposits in a retarding basin might not be satisfactory. It is often impossible to tell when the analyses are made what use will be made of the data at a future date. In some situations, at least, more than one method would be indicated by the probable uses of the data. Of course, if ample funds were always available, the thing to do would be to have all analyses made by both methods, but this is often impractical.

At the present time, we probably cannot see clearly what should be done in each case, but we must go ahead and make the analyses now, in order to have the data when it is needed. Since we must proceed rather blindly, we should at least study the situation and make the best possible

* Consulting Hydraulic Engineer, Branch of Design and Construction, Bureau of Reclamation, Denver, Colo.

decision in the light of our present knowledge. Future study may modify the writer's viewpoint somewhat, but, at present, it appears that there are probably two main classes of cases of which the two examples just given are typical. One class will be the cases involving problems of sedimentation of the finer particles, the silts and clays, and possibly the movement of very heavy concentration such as occur in some rivers and give rise to density currents. For this class of problem, the analyses in native water and at the natural concentration would be needed. The other class includes problems involving comparisons between sediment transported in flowing water and sediment which has been deposited as previously-described.

How should a decision be made on which of these methods of analysis should be used in any given case, or whether both should be used? In some cases, it is fairly evident that both should be used. For example, on the Yung Ting River in China, on which the retarding basins will probably soon be built, both of these methods should be used, a record by both methods being secured, at least during floods. The Rio Puerco in New Mexico is a similar case. When there are sediment flows which might produce density currents, the sediment should be analyzed by both methods. This may sound like a great increase in the sediment program but really it is not, for it involves only a few streams and these only part of the time. In most streams in this country, the size data should be on the basis of the ultimate particle size, at least in the present state of our knowledge and finances.

BILLY T. MITCHELL.* I want to take this opportunity to comment on the use of the bottom withdrawal tube, particularly in relation to what we have found in the Omaha District. I first want to corroborate, with all the emphasis I can express, the opinion of Dr. Howard, Mr. Nelson, and Mr. Lane that the bottom withdrawal tube is probably the most satisfactory instrument that we have at present for determining particle size, and anything that I may say which would tend to refute what has been said up to now is not so intended. My remarks are intended only to show some of the ramifications in the use of this tube, or, for that matter, any sedimenting system whether it be the hydrometer method, the decantation process, or some other method, and to emphasize that the ramifications I will mention will be found in any process. I feel that since we have used the bottom withdrawal method so much in the Omaha District, we at least know whereof we speak. As nearly as I can remember at the moment, we have run some 3,500 tests with the bottom withdrawal tube in the past two years. I don't mean to disparage the number that Dr. Howard has run, and I wouldn't be surprised if the Albuquerque District has approximated, if not exceeded, the number we have made. We intend to continue its use for every purpose that we can make it serve, but we are not satisfied with some of the actions we find in it.

Originally we used the hydrometer method, but the trouble of having too high a concentration (which Dr. Howard and the other gentlemen pointed out) made it seem nearly useless to continue with the hydrometer when we could get so much better results with the bottom withdrawal tube. A sedimenting system in which a hydrometer can operate requires so high a concentration of sediment that flocculation by mass attraction is nearly certain. This phenomenon has been studied to a considerable extent by German scientists, and Tuorila is probably the best authority on this subject, as far as I know. He maintains that, no matter what the suspension medium is - - that is, what type of fluid is used - - mass attraction between particles which are neutral in electrostatic charge will occur if they are close enough to one another. This distance is defined in terms of the ratio of radii of the large and small particles, and where the larger particles settle through the fluid faster than the smaller ones, they will drag the smaller ones along and thus produce flocculation.

We are not worried too much about flocculation due to mass attraction where we use the bottom withdrawal tube because, as Dr. Howard pointed out, the samples can be split. We do split our samples to a concentration within the range for which the bottom withdrawal process was designed. But we are concerned with the type of flocculation which is caused by electrolytic changes of the charges on sediment particles due to the type and amounts of ionized chemical salts in natural waters. The Omaha District has been conducting a preliminary study of the magnitude of this problem for the last few months in the manner described in the following paper.

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* Omaha District, Corps of Engineers, Department of the Army, Omaha, Nebr.

Influence of Water Quality on
Flocculation of Suspended Sediment

by Billy T. Mitchell and Leslie A. Brown *

Introduction

The new science of sedimentation is largely dependent upon the proper concept of sediment particle sizes. Although the determination of size gradation was the concern of most of the pioneers in this field, the interest was largely academic until the expansion of public works within the last fifteen years forced various agencies to consider the best means by which enormous investments in reservoirs, irrigation systems, floodways, navigation channels and other undertakings to control and utilize sediment-laden waters could be protected from the sedimentation hazard. Such means cannot be devised without an adequate knowledge of the behavior of all sediment, which in turn is dependent upon the different behavior of various sizes of sediment particles. Thus, considerable emphasis recently has been given to research directed toward development of accurate methods of determining sediment particle size. Nearly all methods applicable to the analysis of suspended sediment are based on an application of Stokes' Law of settling velocities of spherical quartz particles. The work of the Interdepartmental Committee 1/ has resulted in the invention of the bottom withdrawal tube to analyze sediments ranging in diameter from 0.001 mm. to 1.0 mm. and in concentrations from 300 to 10,000 parts per million. According to this Committee, the hydrometer and decantation processes still must be employed for analyses of materials finer than 1/16 mm. when the concentrations are in excess of 10,000 parts per million unless the samples can be split sufficiently to allow analysis in the bottom withdrawal tube. Any process for analysis of suspended sediment particle size by the application of Stokes' Law is subject to a number of errors. One of the most comprehensive discussions of these errors is given by Krumbein 2/, who indicates that the principal errors lie in the variation from true sphericity and smoothness of the particles, the diameter of the vessel containing the sedimenting system, the tendency toward mutual interference of particles, and the rapidity with which the particles reach a terminal velocity. Most of these factors have been studied by numerous workers to a sufficient degree that corrective measures are available. Thus, Krumbein 2/ discusses the Rubey, Wadell, Oseen and Goldstein formulas which make allowance for shape and surface character of particles. He also states that Lorentz, Landenburg and Arnold investigated wall effects, and that Allen and Arnold computed the maximum size of particle which would not show inertia effects in a given liquid. But he concludes that much too little is known about the mutual interference of particles, except that it is better to have dilute suspensions, preferably less than 25 grams of solid to the liter.

There is another, and perhaps greater, source of error in the application of Stokes' Law to suspended sediment size analysis. This is the possibility of flocculation of particles, not alone because of their proximity to each other, but because of charges on them. Such charges vary with the kind and amount of ions in the water surrounding the particles. Very little has been written on the suspended sediment phase of the subject. Nearly all investigations of flocculation have been made in connection with soils and geological studies, where the samples were in massive form and had to be placed in suspension in the laboratory. It has been recognized for some time that sodium oxalate ($Na_2C_2O_4$) is a good deflocculent, although there are other chemicals which will produce the same effect. It is not generally recognized, apparently, that all known deflocculents have a wide variation in efficiency if either too much or too little is used. Krumbein 3/ found that sodium oxalate had a dispersing effect over a fairly wide range of solution strength on one particular clay, but it too became less effective in dispersing the material if too much or too little was added.

It appears that the problem of flocculation is not one for which there is a universal panacea. Since most of the research on flocculation has been performed by soils scientists and geologists principally as a means of improving soils analyses in laboratories, many of the results thus obtained are not strictly applicable to laboratory analysis of suspended sediment. But flocculation of suspended sediment also has many other ramifications. If the action of chemical dispersing agents is essentially electrolytic, then may not electrolytes in natural waters affect sediment flocculation? Puri and Keen 4/ concluded that the effects of electrolytes on suspensions are not abrupt but cause a continuous change in the degree of dispersion as the electrolyte concentration is varied. They found that the common compounds sodium chloride (NaCl) and potassium chloride (KCl) (which are usually present in natural waters) produced flocculation

* Omaha District, Corps of Engineers, Department of the Army, Omaha, Nebr.
1/ 2/ 3/ 4/ See "References" at close of comments.

with relatively small concentrations and had decreasing dispersing effect with increasing concentration above a certain amount. Krumbein found the same effect with sodium carbonate(Na_2CO_3). Accordingly, is it not possible that suspended sediment may vary in its degree of flocculation as it is carried downstream in water which is varying in its chemical quality? Do different types of sediment react differently with the same water? In that event, is it desirable to have a uniform technique for determining particle size in the laboratory, with the possibility that vital data usable, for example, in estimating the potentiality of density current formation in a reservoir would be completely obscured? It appears that the more logical approach would be to conduct experiments in the field to ascertain the likelihood that varying degrees of flocculation are present in waters of a given chemical quality, and then prepare a manual for normal laboratory use based on the chemical composition of the sediment and the water at the time a sample was obtained. It might be quite worthwhile to equip a mobile sediment laboratory for such a purpose.

It was with such questions in mind that the following experiments were undertaken. It seemed necessary to attempt an evaluation of the magnitude of the flocculation problem in terms of the equipment and techniques now available. The purpose of this paper is to present the problem for general discussion, to furnish data obtained in a preliminary study, and to draw tentative conclusions therefrom.

Basis of Experiment

During the calendar year 1946, a number of samples of water were obtained from the Missouri River at Elbowoods and Garrison Dam Site, North Dakota, and Oahe and Fort Randall Dam Sites, South Dakota, to make chemical analyses of quality. It was noted that the amounts of various radicals varied considerably at the four locations at different times of the year. Since there was no apparent reason for 1946 being an abnormal year with respect to water quality or sediment content, it was decided that the maximum and minimum values given by the analyses for the important radicals during the open-water season at the latter three stations would be adopted for the flocculation study.

In order to have enough identical material to make a considerable number of tests, a special sample of sediment (designated as FT) was obtained from a backwater area along the Missouri River near Sioux City, Iowa. Since this sample contained a considerable amount of coarse material, it did not appear to be the most suitable specimen with which to experiment. Consequently, a second sample (designated as FS) was obtained by salvaging sediment which had settled out of suspension in extra "quality of water" samples, and compositing all salvaged material. This sample was considerably finer than the first, and seemed to offer a better opportunity to observe flocculation effects. All tests described in this paper were conducted with the FS material.

There seemed to be no logical reason to make the required particle size analyses by any method except the bottom withdrawal tube process, since the flocculation effects which would be of sufficient importance to produce errors in ordinary work would have to be apparent in that process. Six tubes of about 534 ml. capacity were selected and assigned exclusively to this experiment. Analyses were made in the routine manner, which is patterned after the process proposed by the Interdepartmental Committee 1/.

It was considered that the sediment sample must be washed thoroughly with distilled water to eliminate practically all natural salts which were in the original water covering the sample or on the sediment particles themselves, since it was only in that manner that a true picture of the effects of controlled salts content could be obtained. Much of the sample was so fine (FS 49, Figure 1) that an absurdly long time delay would have been required to allow the sediment to settle out completely after a washing. Filtering was impracticable because the filter pores would clog with or pass fine particles, and some of the material would be lost with each washing. Consequently, a supercentrifuge was used to clarify the wash water and salvage the finer sediment. A cellulose acetate liner was placed in the supercentrifuge bowl, and the accumulated sediment was washed off this liner back into the sample after each clarification. The residual sales content after each washing was determined by evaporating a specific volume of the clarified water and weighing the residue. All duplicate tests designated by one test number were made upon split samples. After the material deposited on the centrifuge bowl liner was washed back into the sample, the storage jug was well shaken and a small amount of the suspension was poured into a pint jar. The shaking and pouring process was repeated a number of times to insure uniformity of sediment in the sample placed in the jar. The sample in the jar was split into 32 portions by using a microsplitter. Analyses were made with as many of the 32 portions as required, the other portions being poured back into the storage jug. Six washings and analyses were performed before any tests with salts were made. During the course of the experiment, occasional tests of the

1/ See "References" at close of comments.

Character of Tests

It was found that the principal radicals in the Missouri River waters during the open water season of 1946 varied, in parts per million, as follows: sodium (Na), 95 to 25; calcium (Ca), 95 to 32; magnesium (Mg), 32 to 6; potassium (K), 12 to 1; sulfate (SO4), 420 to 130; bicarbonate (HCO_3), 190 to 110; and chloride (Cl), 15 to 1, as shown on Figure 2. Somewhat reduced maxima were used for calcium and sulfate, since they seemed more typical of the open water season. It was decided to make a number of tests using only a single salt in each test, and determining the effect of maxima and minima values of each important positive ion alone. This necessitated, of course, that there also be anions, but the controlling factor was the amount of the cation being tested. Stock solutions of calcium sulfate ($CaSO_4$), magnesium sulfate ($MgSO_4$), sodium sulfate (Na_2SO_4), and sodium bicarbonate ($NaHCO_3$) were prepared in such concentrations that 1 ml. of the solution diluted to 534 ml. (the volume of the bottom withdrawal tube) gave a solution with a known salt concentration in parts per million. The required amount of stock solution for a given test was added to a split portion of the sediment sample, diluted to a total volume of 450 ml. in a pint jar, stirred, and allowed to stand overnight. The next morning the contents of the jar were washed into a bottom withdrawal tube, diluted further to the required final volume of 534 ml., and the analysis begun. This entire group of tests was duplicated, using distilled water which had been boiled to remove carbon dioxide, to determine the effect of dissolved carbon dioxide on the flocculating action of the salts. The washed sediment analyzed alone as controls before and after this series of tests (FS 6 and FS 19) had total dissolved solids of about 1 ppm. when diluted in the bottom withdrawal tubes. Table 1 shows the important characteristics of this series:

TABLE 1.

Test No.	Character of water used in suspension	Tested		Attendant	
		Cation	ppm.	Anion	ppm.
FS 7	Distilled Boiled	Calcium	80	Sulfate	193
FS 8	" "	"	32	"	77
FS 9	" "	Magnesium	25	"	100
FS 10	" "	"	5	"	20
FS 11	" "	Sodium	96	Bicarbonate	255
FS 12	" "	"	24	"	64
FS 13	Distilled Regular	Calcium	80	Sulfate	193
FS 14	"	"	32	"	77
FS 15	"	Magnesium	25	"	100
FS 16	"	"	5	"	20
FS 17	"	Sodium	96	Bicarbonate	255
FS 18	"	"	24	"	64

The results of the tests are shown on Figure 3.

The second section of this single-salt test was designed to determine whether flocculation effects were being produced by the positive or negative radical. Since very little flocculation was caused by sodium bicarbonate (FS 11, 12, 17 and 18), sodium sulfate was used as a substitute for the calcium sulfate and magnesium sulfate used in Tests FS 7, 8, 9, 10, 13, 14, 15, and 16, holding the amount of sulfate approximately equal for each individual comparison. Table 2 shows the test conditions:

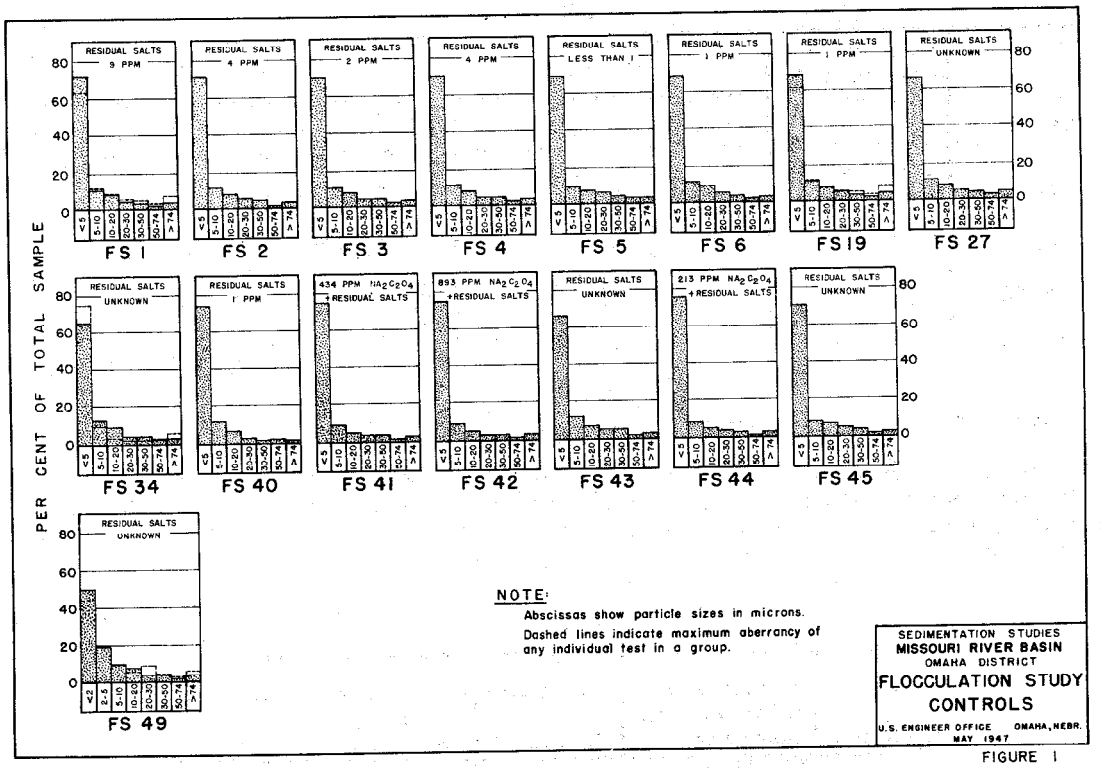


FIGURE 1

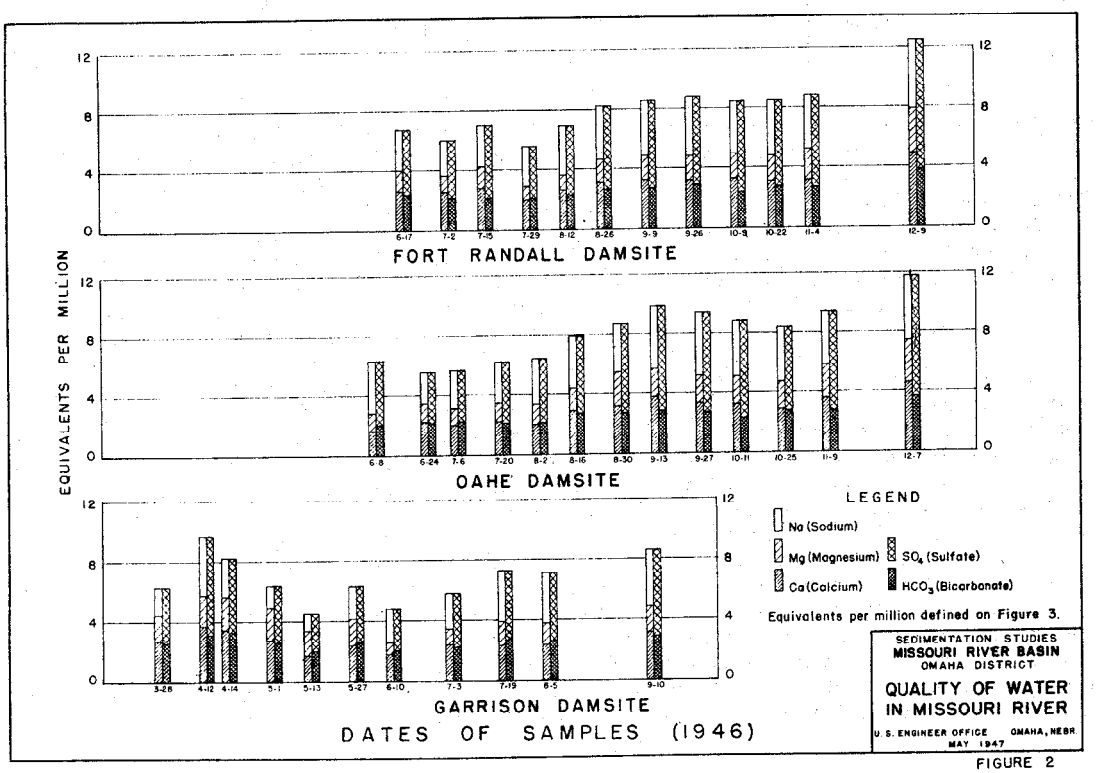


FIGURE 2

TABLE 2.

Test No.	Character of water used in suspension	Present test				Previous test			
		Cation	ppm.	Anion	ppm.	Anion	ppm.	Cation	ppm.
FS 20	Distilled Boiled	Sodium	96	Sulfate	200	Sulfate	193	Calcium	80
FS 21	"	"	48	"	100	"	100	Magnesium	25
FS 22	"	"	38	"	80	"	77	Calcium	32
FS 23	"	"	10	"	20	"	20	Magnesium	5

The results of these tests are shown on Figure 3.

The third section of the single-salt tests was intended to compare the effect of calcium, magnesium and sodium sulfates on flocculation when the sulfate radical was at about the maximum concentration found in the Missouri River during the open-water season. It was performed as shown in Table 3.

TABLE 3.

Test no.	Character of water used in suspension	Attendant		Tested	
		Cation	ppm.	Anion	ppm.
FS 24	Distilled Boiled	Calcium	129	Sulfate	309
FS 25	"	Magnesium	75	"	300
FS 26	"	Sodium	144	"	300

The results of this test are shown on Figure 3. FS 27, a control test with the washed material only, showed no appreciable change over similar tests FS 1 to 6, inclusive, and FS 19 (Figure 1).

The portion of the experiment using multiple salts consisted of a series of particle-size analyses in suspensions to which various amounts of calcium sulfate, magnesium sulfate, and sodium bicarbonate were added in order to simulate the ranges of dissolved solids conditions which were found to exist in the Missouri River in North and South Dakota during the open-water season of 1946. It was found advisable to repeat certain of the tests to verify the variations between the quadruplicates of the first tests. All tests were performed in suspensions of boiled distilled water. The first three tests, FS 28, 29 and 30, were designed to provide the maximum calcium and magnesium cations found in the quality of water tests and to vary the sodium bicarbonate content between maximum and minimum values. The second group of three tests, FS 31, 32 and 37, were made using a moderate amount of calcium and magnesium cations, with variation of the sodium bicarbonate content between the maximum and minimum. The third group of three tests, FS 33, 39 and 48 were made using the minimum amount of calcium and magnesium cations, again with variation of the sodium bicarbonate content between maximum and minimum. Tests FS 35, 36 and 47 duplicated Tests FS 31, 32 and 39 respectively, and Tests FS 38 and 46 duplicated Test FS 33. Tests FS 34, 40 and 45 were made as controls, using the washed sediment without any added salts, and having residual salts of only about 1 ppm. in the bottom withdrawal tube. It was found that a calcium carbonate precipitate formed when calcium sulfate and sodium bicarbonate were mixed, and for that reason, the ionized concentrations of calcium and bicarbonate radicals were less than those desired. Table 4 shows the amounts of the various ions in solution for the different tests, initially and after precipitation had occurred as determined by preparing separate solutions and filtering off the precipitate. The parenthesized figures are the amounts remaining in solution:

TABLE 4.
(Ions in ppm.)

Test no.	Calcium	Magnesium	Sodium	Sulfate	Bicarbonate
FS 28	80 (61)	25	96	293	255 (196)
FS 29	80 (66)	25	72	293	191 (147)
FS 30	80 (80)	25	24	293	64 (64)
FS 31 & 35	56 (31)	15	96	195	255 (176)
FS 32 & 36	56 (40)	15	72	195	191 (141)
FS 37	56 (56)	15	24	195	64 (64)
FS 33 & 38	32 (7)	5	96	97	255 (180)
FS 46	32 (18)	5	96	97	255 (212)
FS 39	32 (12)	5	72	97	191 (131)
FS 47	32 (22)	5	72	97	191 (161)
FS 48	32 (30)	5	24	97	64 (58)

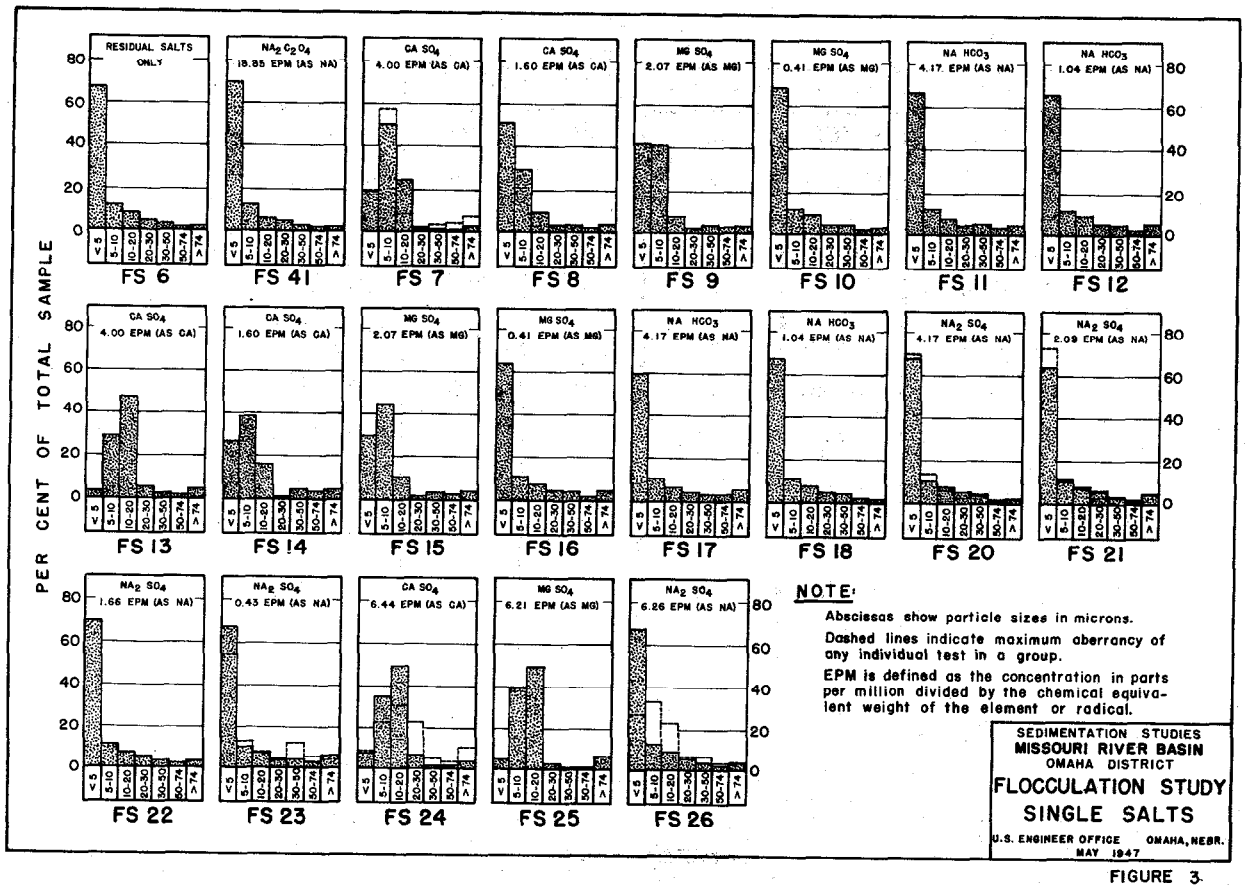
Figure 4 shows the results of these tests.

In order to determine if the original washed sample was at its maximum deflocculation, a series of four tests in **quadruplicate** were made in boiled distilled water, using various amounts of sodium oxalate ($Na_2C_2O_4$) as a dispersing agent. It is the practice in many laboratories to use 15 cc. of a 1.5 percent solution of sodium oxalate in all tests for particle size. This series, therefore, was prepared using 7.5 cc., 15 cc., and 30 cc. of this solution in Tests FS 41, 42 and 44 respectively. FS 43 was run as control, using no salts of any kind. It was found desirable, because of the disparity between the quadruplicates, to repeat the series, again in quadruplicate. The mean particle-size distributions thus obtained from eight tests each are shown on Figure 1 for comparison with the untreated sediment tests.

A series of chemical tests also were run to determine the importance of chemical precipitation in the application of corrections for dissolved salts to the weights of sediment fractions withdrawn from the bottom withdrawal tube. The results of these tests are described in a later paragraph.

In addition to the bottom withdrawal analyses of sediment and the chemical tests of waters used in the experiments, a chemical and petrographic analysis was made of the sediment used throughout the experiment. The following results were obtained from the chemical analysis:

Compound	Percentage of total sample
Silica (SiO_2)	56.10
Ferric Oxide (Fe_2O_3)	5.39
Alumina (Al_2O_3)	21.33
Calcium Oxide (CaO)	2.43
Magnesium Oxide (MgO)	2.28
Sodium Oxide (Na_2O)	0.60
Potassium Oxide (K_2O)	2.02
Phosphorus Pentoxide (P_2O_5)	0.23



NOTE:

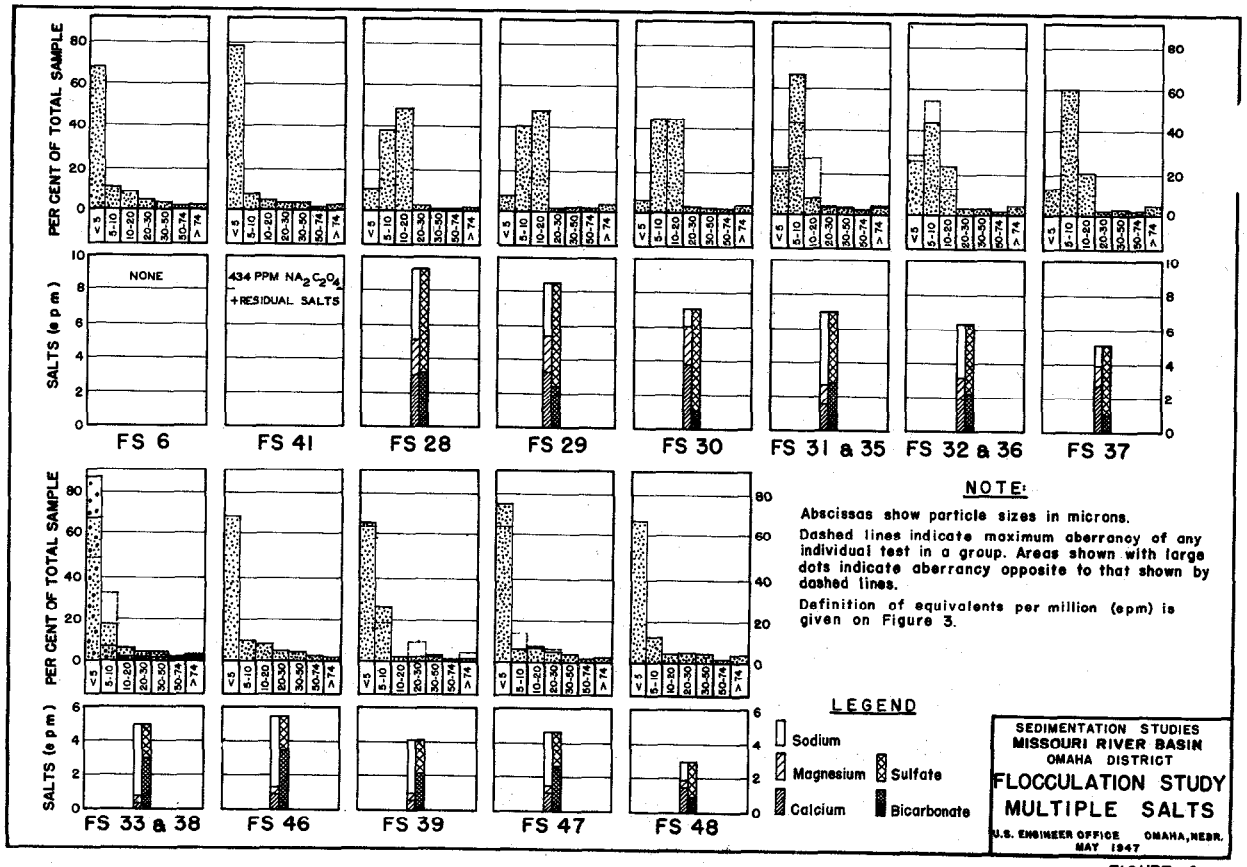
Abscissas show particle sizes in microns.

Dashed lines indicate maximum aberrancy of any individual test in a group.

EPM is defined as the concentration in parts per million divided by the chemical equivalent weight of the element or radical.

SEDIMENTATION STUDIES
MISSOURI RIVER BASIN
OMAHA DISTRICT
FLOCCULATION STUDY
SINGLE SALTS
U.S. ENGINEER OFFICE OMAHA, NEBR.
MAY 1947

FIGURE 3



NOTE:

Abscissas show particle sizes in microns.

Dashed lines indicate maximum aberrancy of any individual test in a group. Areas shown with large dots indicate aberrancy opposite to that shown by dashed lines.

Definition of equivalents per million (epm) is given on Figure 3.

LEGEND

Sodium
Magnesium
Calcium
Sulfate
Bicarbonate

SEDIMENTATION STUDIES
MISSOURI RIVER BASIN
OMAHA DISTRICT
FLOCCULATION STUDY
MULTIPLE SALTS
U.S. ENGINEER OFFICE OMAHA, NEBR.
MAY 1947

FIGURE 4

Compound	Percentage of total sample
Carbon Dioxide (CO_2)	1.29
Water (expelled at 1000 °C)	6.22
Organic Matter (as Carbon)	0.68
Manganese Oxide (MnO)	Not yet determined
Total	98.57

The petrographic analysis is quoted "Brownish material . . . . shows aggregate polarization present in the large chunks. Small pieces of quartz, carbonate, chert, feldspar, and muscovite."

Results of Tests

The comparison of control tests shown on Figure 1 demonstrates the difficulty of maintaining exact uniformity of the test material over a period of time. It was found that most of the residual salts could be removed by washing in regular distilled water, but that the salt content would rise considerably after the sediment stood for a few days (FS 3 and 4). It appeared that carbon dioxide in this water was combining with calcite to form an ionized calcium bicarbonate. All washings thereafter were made with distilled water which had been boiled to drive off the excess carbon dioxide (FS 5, 6, 19 and 40), with the result that the sediment stood for nearly one month without gaining more than 14 ppm. of salts in the supernatant water, and an additional month during which a loss of 10 ppm. of salts occurred. The increased clay content in FS 40 may be due primarily to a slight disintegration of calcite-cemented particles with no concurrent gain of salts in solution. This conclusion may be invalid, however, since FS 43 has a slightly lower clay content. It is apparent, in spite of these minor variations, that the test material was maintained in essential uniformity throughout the entire study. It should be noted also that the dilution of the small samples in the bottom withdrawal tube reduces the residual salts to a negligible figure of 1 ppm. to 4 ppm.

The effect of sodium oxalate on the test sediment is to increase the dispersion slightly. However, the size distributions using 15cc. and 30cc. of sodium oxalate (FS 41 and 42) are practically identical, are only 8 percent higher in clay content than the untreated sample FS 43, and are only 4 percent higher in clay than the freshly-washed sediment in FS 40. There is a somewhat lesser dispersing effect when only 7.5cc. of sodium oxalate is used. It is apparent, however, that the original washed and untreated sediment is somewhat flocculated, although the condition is not sufficiently serious to invalidate the experiments where various salts were used. Incidentally, it appears that sodium exerts a greater dispersing effect when present in an oxalate compound than in carbonate or sulfate compounds (FS 11, 26 and 41), although a certain moderate sodium sulfate concentration tends to approach the sodium oxalate action (FS 22 and 41).

The results of the single salt tests given on Figure 3 show that calcium alone (FS 7 and 8), in both the maximum and minimum quantities normally found in the river, will cause flocculation of the clays in the test material into particles of a fine silt size, although the magnitude of the action is much greater with the higher calcium content than with the lower. Flocculation also occurs when the maximum amount of magnesium alone is present (FS 9), although the action is not as great as with the maximum amount of calcium. The minimum amount of magnesium alone apparently does not cause flocculation (FS 10). Sodium, when added as a bicarbonate, seems to cause a slight flocculation (FS 11 and 12), but in the maximum case much less than with the calcium and magnesium maxima (FS 11, 7 and 9).

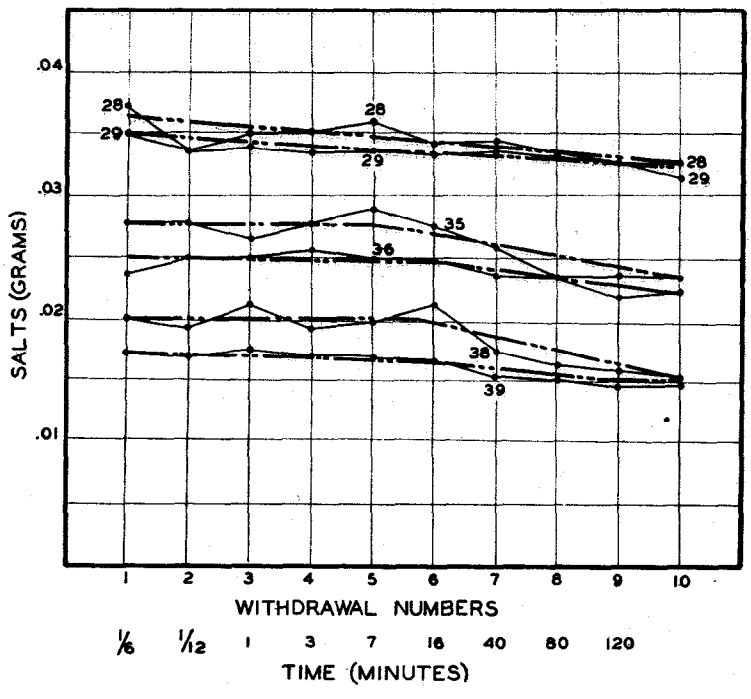
The preceding series of tests, when run in distilled water which had not been boiled to reduce the content of carbon dioxide, shows considerably greater flocculation when maximum and minimum calcium and maximum magnesium are used (FS 7 and 13, 8 and 14, and 9 and 15). There is relatively little difference due to the presence of carbon dioxide with minimum magnesium and with both maximum and minimum sodium (FS 10 and 16, 11 and 17, and 12 and 18).

The tests to determine whether the positive or negative radicals were instrumental in causing flocculation demonstrated that the positive radicals were responsible. By substituting sodium sulfate for the maximum and minimum amounts of calcium sulfate and magnesium sulfate, the insignificant effect of sulfate is seen in the comparisons of FS 7 and 20, 9 and 21, and 8 and 22. Comparison of FS 10 and 23, however, indicates that sodium sulfate causes slightly more

flocculations than magnesium sulfate when each is present in small amounts. It also appears that the effect of sodium sulfate on the clay fractions of this material is somewhat erratic (FS 20, 21, 22 and 23).

The comparative effects of calcium, magnesium, and sodium, when the corresponding sulfate was near the maximum concentration generally found in the Missouri River, are shown by tests FS 24, 25 and 26. It is interesting to note that, although calcium and magnesium show much greater flocculating effects than sodium, magnesium has slightly more effect on flocculating action than calcium on an equivalent basis.

The results of the multiple-salts tests using a combination of calcium sulfate, magnesium sulfate, and sodium bicarbonate are shown in Figure 4. For purposes of comparison, the amounts of salts in the suspensions in chemical equivalents per million are also given by bar diagrams. Precipitation of calcium carbonate prevented maintenance of a uniform amount of calcium in the suspensions while the sodium bicarbonate content was varied, so that a comparison of the flocculating action of the cations alone is not possible. Presence of the precipitates required that non-uniform correction of the dissolved salts for each fractional withdrawal be determined, even though the weights of such precipitates were negligible compared to the weight of sediment in the withdrawals containing those precipitates. The corrections were determined by making bottom withdrawal tests of solutions identical to those used in Tests FS 28, 29, 31 and 35, 32 and 36, 33 and 38, and 39 with the sediment omitted. The results and corrections are shown on Figure 5.



SALTS PLACED IN SOLUTION (P P M)

TEST NO	CA	MG	NA	SO_4	HCO_3
FS 28	80	25	96	293	255
FS 29	80	25	72	293	191
FS 35	56	15	96	195	255
FS 36	56	15	72	195	191
FS 38	32	5	96	97	255
FS 39	32	5	72	97	191

NOTES:

1. NO SEDIMENT USED IN THESE TESTS.
2. ALL POINTS ARE AVERAGES OF DUPLICATE TESTS.
3. DASHED LINES ARE CORRECTIONS APPLIED TO RESPECTIVE TESTS WHERE SEDIMENT WAS USED.

SEDIMENTATION STUDIES
MISSOURI RIVER BASIN
OMAHA DISTRICT
FLOCCULATION STUDY
VARIATION IN SALTS
(BOTTOM WITHDRAWAL PROCESS)
U.S. ENGINEER OFFICE OMAHA, NEBR.
MAY 1947

FIGURE 5

No visible precipitates were found solutions identical to those in Tests FS 30 and 37. Portions of these solutions were simply evaporated and the residues were weighed. The discrepancy in weights between the residues and the original salts added showed that chemical recomposition was occurring even at the low temperatures used for evaporation. In order to determine the reactions taking place during the evaporation of the solutions for Tests FS 30 and 37, solutions of various concentrations of the individual salts used for those tests were evaporated to dryness. Although the loss of carbon dioxide from the solution of sodium bicarbonate and the gain of water of crystallization during evaporation of the calcium and magnesium sulfate solutions did not vary in the same ratios as the concentrations of those salts, it was possible to calculate the recompositions occurring during evaporation of FS 30 and 37, as well as all other tests using multiple

salts. In addition, these evaporations of individual salts solutions gave the proper corrections to be applied in the sediment tests using individual salts alone. FS 46 and 47, the reruns of Tests FS 33 and 38, and FS 39, made because of the fluctuation in flocculation between duplicates of those tests, were analyzed in suspensions from which the precipitated calcium carbonate had been removed by filtration. FS 48 was run in a similar manner.

It is apparent that the multiple-salts tests in which calcium and magnesium are present in large amounts are more highly flocculated than those in which the amounts of these cations are smaller. It seems that the size distribution approaches that of the original untreated sample when the total calcium and magnesium content is somewhat less than 2.75 equivalents per million (55 parts per million in terms of calcium only). It does not appear that the amount of sodium, sulfate or bicarbonate radicals have an appreciable effect on flocculation when the calcium and magnesium are present in sufficient quantity.

Conclusions

A number of further tests are indicated by the results thus far and are being planned at present. It appears, however, that a point has been reached where some tentative conclusions can be drawn and placed before the engineers engaged in sediment work for their consideration and suggestions.

First, and most important, it appears evident that apparent sediment particle-size may vary considerably if the suspensions used for analysis contain appreciable and different amounts of calcium or magnesium salts. Although a dispersing agent such as sodium oxalate may be effective in counteracting the tendency toward flocculation in the analysis, there is no assurance that such counteraction has reproduced the saline conditions under which the sediment originally was being transported by the stream. In fact, there is relatively small probability that it does. Even if a dispersing agent will always insure that the basic grain-size distribution is obtained, of what avail is that unless the effective size distribution with each important variation in the chemical constituency of the suspension is known? It appears entirely possible, for example, that the formation and maintenance of sediment density currents in large reservoirs depends to a considerable extent on the flocculating or dispersing action of electrolytes in the river and the lake waters. It also appears that only through experimental analysis of sediment particle sizes and of water quality at the site of sediment sampling, probably also at different seasons of the year, the true relationships can be discovered.

The formation of precipitates in the artificial waters used in the latter series of tests seems to point toward a similar situation in natural waters. The tendency toward greater flocculation in Tests FS 13, 14 and 15 then in 7, 8 and 9 indicates that the greater amount of dissolved carbon dioxide in the former group permitted the presence of more calcium and magnesium ions in the suspension. The presence of more dissolved calcium and magnesium in natural waters during cold weather is also partly explainable by the facts that cold water can retain more free carbon dioxide than warm water, and that carbon dioxide must be present in a solution in order to have ionized calcium bicarbonate instead of the nearly insoluble calcium carbonate. The decrease of carbon dioxide in the stream must cause precipitation of calcium carbonate (calcite); and it is probable that an increase in carbon dioxide content causes solution of some calcite. The latter assumption was corroborated during this study by the necessity of using boiled distilled water for washing the sample to prevent increase in the dissolved solids content of the water on the sample during a lapse of time.

The tests reported in this paper were not of sufficient scope to permit definite conclusions concerning the exact nature of flocculation to be drawn. The most important factor yet to be evaluated is the base-exchange capacity of the sediment being used in this study. This characteristic is being determined at present, but difficulties are being encountered which are due probably to the amount of calcite ($CaCO_3$) in the sediment. The chemical analysis of the sediment shows only a small percentage of calcium oxide and carbon dioxide, but if these compounds exist as as calcium carbonate, it is probable that the quantity is sufficient to affect the determination of base-exchange capacity made by the ordinary methods. When the final determination is completed, it should be possible to form a fairly definite conception of the action of various salts on this sediment. Until then, only conjecture is possible.

The petrographic analysis shows that small pieces of quartz, chert, carbonate, feldspar, and muscovite were identified, although the majority of the material was unidentifiable. Since quart quartz and chert are silica (SiO_2), carbonate is probably calcite ($CaCO_3$) or magnesium carbonate ($MgCO_3$), feldspar is probably orthoclase ($KAlSi_3O_8$) or plagioclase ($NaAlSi_3O_8$ to $CaAl_2Si_2O_8$) and muscovite is white mica ($KAl_2Si_4O_{10}(OH)_2$), the various compounds shown in the chemical

analysis undoubtedly are found partly in these minerals. Because the balance of the material in the sample could not be resolved microscopically, it is probable that it is clay. Its mineral composition is unknown, but since it was obtained from river water, and consequently from widely different areas, the minerals probably include the three principal types: montmorillonite, illite, and kaolinite. It is probable, from the presence of about 2 percent of K_2O in the chemical analysis, that the clays are somewhat micaceous, although this proportion of potassium could result from the presence of muscovite or orthoclase in the sample. Otherwise the material has the general chemical composition of montmorillonite. Ross and Hendricks 5/ state that hydrous micas (illite), which consist of mixed layers of montmorillonite minerals and micas, occur widely, particularly in arid regions. They also state that the mineralogy of these mica-like minerals has not been adequately studied. If the sample utilized in this study consists principally of a mixture of montmorillonite and illite, it may be quite difficult to obtain an adequate base-exchange capacity.

In a preceding paragraph, it was stated that the particle size distribution of this sample appears to show flocculation only when the water content of calcium and magnesium exceeds about 50 to 55 parts per million as calcium. If this assumption is correct, flocculation is present to some degree in much of the water in the Missouri River basin. Since serious flocculation was found when the calcium and magnesium content was greater than about 80 to 100 parts per million as calcium, it seems that this situation is frequently present in tributary streams and to a lesser extent in the Missouri River itself.

The discrepancies that may be produced by analyzing sediment samples in distilled water can also be envisioned if the results thus far obtained in this study can be duplicated with other sediments. When a sample of sediment taken from the natural stream which contains over 50 parts per million of calcium and magnesium is brought into the laboratory, allowed to stand until the supernatant water is clear and is removed and the sediment sample with the remaining water is placed in the bottom withdrawal tube and distilled water added, the salts content is diluted from 10 to 25 times. Accordingly, the calcium and magnesium content of the native water would have to be over 500 parts per million for flocculation to be present in the bottom withdrawal process. Such salts contents do occur at times on tributaries. But in general, the bottom withdrawal analysis should exhibit little or no flocculation when distilled water is used as the dispersion medium. However, this type of analysis gives no idea of the particle size distribution in the natural stream under such conditions. Under such conditions proper use of particle size data for most engineering purposes is dependent upon a thorough knowledge of chemical flocculation characteristics of that particular sediment and of the quality of water in the natural streams and reservoirs in question. It might be well to make frequent laboratory analysis of sediment in native water to provide a better basis of comparison. The entire problem of flocculation would be more nearly susceptible of solution if a thorough study were made in the field with a mobile laboratory.

This study, although of a preliminary nature, has shown the value of investigations into the physical and chemical nature of suspended sediment. The results obtained thus far imply that an adequate understanding of suspended sediment phenomena must include a consideration of the chemical quality of waters and sediment. The techniques now employed in the analysis of suspended sediment particle size may require substantial revision when sufficient knowledge of the aspects of flocculation is obtained. Many authorities on sediment are giving serious consideration to this problem because it appears to have so much importance in the design and operation of reservoirs and river systems.

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