THE SEDIMENT PROBLEM

FLOOD CONTROL SERIES

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The various studies carried out by the Bureau of Flood Control and Water Resources Development of the Economic Commission for Asia and the Far East are being published as *Flood Control Series*. The first number of this series entitled *Flood Damage and Flood Control Activities in Asia and the Far East* was published in October 1950. The second volume of the series entitled *Method and Problems of Flood Control in Asia and the Far East* was published in 1951. The third number, *Proceedings of the Regional Technical Conference on Flood Control in Asia and the Far East*, was published in 1952. The fourth one in this series, *River Training and Bank Protection*, was published in 1953. The present publication, *The Sediment Problem*, is the fifth one of this series.

In a river system as a whole, the sediment problem starts at the very source of sediment supply in the headwater area and continues down the river system. In the course of the travel of the sediment from its source to the sea, it gives rise to several problems particularly when the regime of flow is altered by the confinement of flow with embankments, by the heading-up of water by dams and by the diversion of flow for irrigation etc. This publication, having discussed briefly the extent of soil erosion in this region, deals essentially with the various aspects of the sediment problem after the soil has been removed from the land and has entered natural streams or canals. It covers transportation of sediment, silting and scouring of canals, silting of reservoirs, action of sediment on the regime of rivers and the sampling and analysis of sediment. The report will be useful to workers in the field of sediment engineering. It is presumed that the reader has sufficient background knowledge to interpret the various results because all of the background development leading up to the results cannot be presented in a work of this nature. To follow up the back-ground development in detail, readers are requested go back to the original references which have been given in footnote in all cases.

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THE SEDIMENT PROBLEM

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SYMBOLS

The following symbols have been used in this publication to represent the quantities mentioned against each one of these, unless otherwise stated inside the text of the publication.

A	Area
a	Any specific distance from the bed
B	Surface width of a channel
b	Sub-script for bed
C, C_1, C_2,	Coefficients
C_d	Drag Coefficient
c	Concentration of sediment by wt. or vol. per unit wt. or vol. of fluid
c_a	Concentration of sediment at a distance *a* from the bed
c_b	Percentage of particles of the total river bed material
D	Depth of open channel or diameter of pipe
D_m	Mean depth
d	Diameter of sediment particles
E	Exchange coefficient or coefficient of mixing
F	Force
f	Silt factor
g	Gravitational constant
H	Head of water
H_e	Loss of head of water in friction
J	Percentage of sediment (by number, or volume, or usually by weight) smaller or larger than the corresponding size of sediment which is in fact a summation of the distribution curve
K	A constant, a ratio
l	Length
m	An exponent
N	Manning's roughness coefficient
N_s	Manning's roughness coefficient for no transportation of sediment
n	Number of class intervals in a sediment sample
P	Wetted perimeter
p	Weight of a sediment particle
p_o	Initial weight of a sediment particle
Q	Total discharge of water
Q_s	Rate of sediment transport
Q_{sq}	Rate of sediment transport by any given discharge of water Q
q	Discharge of water per unit width of channel
q_c	Critical discharge per unit width at which sediment movement commences
q_s	Rate of transport of sediment per unit width
R	Hydraulic mean radius
R_e	Reynolds' number
r	Resistance of particles to motion
S	Slope or energy gradient (clear water)
S_c	Slope or energy gradient (mixture of water and sediment)
s_s	Specific gravity of sediment
s	Distance travelled by sediment particles
V	Velocity of flow
V_o	Critical (non-scouring and non-silting) velocity
W	Bottom width of a channel
W_d	Sediment run-off
w	Terminal fall velocity
x, y, z	The co-ordinate axis
α, α_1, α_2,	Coefficients
β	Constant
ϕ	Angle of side slope
θ	Angle of repose
ϕ_1, ϕ_2, ϕ_3	Coefficients
μ	Absolute viscosity
ν	Kinematic viscosity
ς	A dimensionless factor
ε	Friction factor
τ	Tractive force
τ_c	Critical tractive force
γ	Specific weight
ρ'	Density of sediment
ρ	Density of water

INTRODUCTION

The importance of the sediment problem to all countries of the ECAFE region was recognized by the Economic Commission for Asia and the Far East, as early as 1947 at its second session at Baguio, at which the Commission, while considering the establishment of the Bureau of Flood Control, recommended that the problem of silting of river beds may be studied by the Bureau in conjunction with any technical organizations concerned. Subsequently various national and regional organizations also stressed the common importance of this problem.

Sediment deposition is a troublesome process along rivers and streams; it raises stream beds, thereby increasing flood heights and inundation; it piles up sediment in immense quantities behind dams, thereby reducing their capacities and function; it causes the rivers to meander and often to leave their original courses and flow along new courses, devastating vast areas of excellent land; it silts up irrigation and navigation channels making them inefficient; and it creates a variety of difficult problems for the engineer, the agriculturist and the forester.

The main cause of the sediment trouble is accelerated soil erosion. Normal denudation or geological erosion is an important process in soil formation, whereby the original rock material is continuously broken down and sorted out by wind and water until it becomes suitable for plant growth. Plants, by the binding effects of their roots, by the protection they afford against rain and wind, and by the fertility they impart to the soil, bring denudation almost to a standstill. Nevertheless some slight denudation is always occuring. An equilibrium is ultimately reached between denudation and soil formation which is governed by climate, soil, rainfall, topography and vegetal cover. Cultivation, deforestation or the destruction of the natural vegetation by grazing, or other means, unless carried out according to certain immutable conditions imposed by each region, may so accelerate denudation that soil, which would normally be washed or blown away in a century, may disappear within a year or sometimes even within a single day.

After the natural equilibrium has been disturbed, rainfall, that was formerly absorbed by the soil protected by vegetal cover, runs off the bare surface, carrying soil with it, and sheet erosion begins. This gradually gets worse, the run-off collects in rivulets, where its erosive and transporting process are enormously increased, and the rivulets become gullies and the gullies join to become chasms, penetrating through the soil into the barren subsoil.

As a result of erosion, river flow during floods may be loaded to capacity with boulders, gravel and clay. Steep hill torrents, which are known for the violence and irregularity of their discharge, carry large amounts of suspended matter which is deposited on the less steep slopes in a characteristic detrital cone which continually increases in size. These torrents, as they debouch on to the cultivated sandy plains, silt up their original channels and floods are forced over wide areas. As the flood flow progresses downwards, and the gradients flatten, the stream unloads its sediment charge. This unloading process dams up channels, and causes streams to break out of their banks. The streams, then, wander across areas occupied by buildings, crops, orchards, railways, highways, and factories. Lower down their courses, the streams pick up fresh loads of sediment to be unloaded and deposited in dwellings and on alluvial lands. Further, as a result of excessive sediment charge, rivers meander, and sometimes entirely change their courses, like the Yelolw River in China and the Kosi in India, devastating vast areas of fertile land, rendering millions of people homeless and without occupation and food.

In the case of irrigation canals, as the main canals bifurcate and branch out the capacity of the water to transport sediment becomes less and less, both as regards quantity and size of particles, as the canal discharge is reduced. Finally, the small water channels for irrigating fields can transport only some sediment of the finer fractions. This results in the silting of canals, branches, laterals, sub-laterals and water channels, and a very large amount of labour and expense is needed every year to maintain the canal system in efficient working order. The sediment has often to be excavated at huge expense and the areas required for depositing the excavated sediment become an important problem.

There is also huge problem of the silting of reservoirs, on which depends the life of the people who benefit from the water stored, from the power generated and from the flood prevention. In modern planning of reservoir, allowance is made for sediment storage which usually occupies the bottom of the reservoir in front of the dam. Unfortunately, measurements taken at existing reservoirs show that a major portion of the sediment deposit, instead of finding its way to the assigned place of storage, settles in the places designed for live storage, thus greatly affecting the functioning of the reservoir. It is also true that the rate of silting of a reservoir can be estimated quite accurately during planning. But the time that can be allowed for the reservoir to be completely filled up is an economic question which can be decided only by the consideration of the benefits derived from the useful period. The benefits so derived should cover the cost of the reservoir. However, one should not overlook the consequences if any already developed region is once again deprived of the use of water for purposes of drinking, irrigation, power production and for navigation. The intangible loss may be tremendeous.

Thus the problem of sediment is of the greatest importance, not only for flood control, but also for multiple-purpose river basin development and needs the attention of all concerned at the planning stage.

When considering a river system as a whole, it can be said that the sediment problem starts at the very source of sediment supply in the head water area and ends at the river outlet into the sea.[1] In the course of the travel of sediment from its source to the sea, it gives rise to several problems, particularly when the regime of flow is altered by confining the flow by constructing embankments, heading up of flow by constructing dams, and diversion of flow for irrigation etc. The solution of the problem appears to be either the elimination or the reduction of sediment from the very source of supply, and the discharging of the sediment already present in the streams and rivers into the sea. Elimination or reduction of erosion is no doubt the fundamental solution of the problem and this subject is dealt with along with the various methods of watershed management including torrent control under section *Watershed erosion control* of chapter IV. Discharging the sediment load of rivers into the sea, whenever possible, is a subject which comes within the scope of river training and has been comprehensively dealt with in a separate report prepared by the Bureau.[2]

Of practical and immediate importance are, indeed, the various problems on engineering work created by sediment as it travels from its source to the sea. Such problems include the silting of reservoirs, the scouring of river bed below a dam, the change of river bed due to diking and the aggradation of river bed due to withdrawal of clear water flow for irrigation, etc., and basically concern the equilibrium between river slope, cross-section etc. on the one hand, and sediment charge and sediment grade on the other. It is only through a thorough understanding of the individual problems that a comprehensive knowledge of sediment for the development of river basin as a whole can be obtained. Therefore, the main object of the present publication is to consider the various aspects of the sediment problem after the soil has been removed from the land and has entered natural streams.

Chapter I deals with the problem of soil erosion in general, briefly describes the extent of soil erosion in different countries of the region and gives information regarding the sediment content of some important rivers. The figures show that the annual depth of erosion on some rivers like the Kosi in India and the Yellow River in China amounts to nearly 2 mm per annum, while in the case of the Ching river in China, it is as much as 4.86 mm. The annual silt run-off per km^2 of catchment area is the maximum on the Ching river, being 7,190 t (18,622 t per sq mi), Kosi and Yellow River coming next with 2,820 t and 2,640 t (7,304 t and 6,838 t per sq mi) respectively. These figures are very high indeed in comparison with other rivers of the world.

Chapter II deals with the theory of transportation of sediment, and the various formulae which have been developed by different workers for suspended and bed load sediment. After pointing out the relative importance of the two categories of sediment in regard to practical problems, it concludes that any rational formula for the design of channels should include both bed and suspended loads. The important factors that should be included in the equations for transportation of sediment have also been suggested.

Silting and scouring of channels are discussed in chapter III. In view of the fact that the amount and grades of sediment that can be carried by a channel are fixed by the volume of water it carries, its velocity and the available slopes, it has been found that it is desirable to prevent as far as possible the coarser particles from entering canals by providing silt excluders and extractors. Different types of desilting works like extractors and excluders in use in various countries both within and outside the region have also been briefly described. The design of canals, capable of transporting a

1. If, however, the sea is shallow and the current is not able to carry away the sediment discharged by the river, a delta will be formed and the river mouth will extend rapidly into the sea which flattens the river slope and induces aggradation. In such instances, the sediment problem does not end even after the sediment load reaches the sea.

2. United Nations Economic Commission for Asia and the Far East, Bureau of Flood Control, *River Training and River Bank Protection* (*Flood Control Series* No. 4).

certain amount and quality of sediment with a specified discharge, has been pointed out to be essentially a problem of bed load transportation. The only set of formulae which take into consideration the sediment charge was that presented by C. Inglis as a result of his observations in India and Pakistan, but the constants involved yet remain to be determined and the validity of the formulae has yet to be verified. An example has been given showing the possible use of the bed load equations in the design of stable channels and it has been suggested that further work on these lines is essential for a solution of the problem.

The silting of reservoirs has been dealt with in some detail in chapter IV. The importance of the location of sediment in a reservoir has been stressed and the effect of the rate of compaction of sediment on the useful life of a reservoir has been pointed out. Remedial measures that can be adopted to control sediment for prolonging the life of a reservoir have been discussed. It has been stressed that watershed erosion control is the only method which aims at dealing with the problem at its source by preventing the movement of sediment, while all other methods are designed to deal with the sediment after its entry into the river, and thence into the reservoir.

The action of sediment on the regime of rivers is discussed in chapter V. The major effects of sediment on river regime such as aggradation, degradation and meandering are described. The effect of the construction of dams and weirs on river regime both upstream and downstream of the structure is also discussed.

Finally, chapter VI describes the various methods of sampling bed and suspended load and the analysis of the samples.

Needless to say, the sediment problem is a complicated one and sediment engineering, a science dealing with the formation, transportation, deposition, measurement, analysis and treatment of sediment, is still in its infancy. The problem bears special importance to countries of this region where a large number of projects for the development of water resources are being embarked upon and where the streams carry considerable quantities of sediment load. Countries in this region cannot afford to invest any substantial amount in a project like a reservoir, the functions of which would be curtailed considerably in a short period of time. It is imperative that effective measures should be taken before launching upon new projects, to combat the constant menace of sediment that must have its effect on such projects.

While the governments of various countries are alive to the seriousness of the sediment problem, little seems to have been done to meet the situation, considering the immensity of the task. The present publication, having summarized and analysed the development of sediment engineering up to date,[1] will have served its purpose if it helps to stimulate further work on the sediment problem in countries of this region.

Although the sources from which information has been compiled for this publication have been freely acknowledged by references in the text, the valuable comments and suggestions, which led to the final shape of this publication, received from the following learned engineers are gratefully acknowledged in particular: Professor E. W. Lane, Hydraulic Engineer, Denver, Col. USA; Professor Vito A. Vanoni, California Institute of Technology, Pasadena, Calif., USA; Professor S. Hayami, Kyoto University, Kyoto, Japan; Mr. L. N. McClellan, Chief Engineer, and other engineers of the US Bureau of Reclamation, Denver, Colorado.

1. In additin to the references given inside the text of this publication, readers are referred to "Annotated Bibliography on Sedimentation", *Sedimentation Bulletin* No. 2, February 1950, compiled under the auspices of the Sub-Committee on Sedimentation, Federal Inter-Agency River Basin Committee of the United States.

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Chapter I

SOIL EROSION IN VARIOUS COUNTRIES OF THE REGION

GENERAL

As it is, erosion is a beneficial natural process without which the world could not have existed. All the rich fertile alluvial plains are formed by the process of erosion and sedimentation and nature, if not interfered with, adjusts this process in such a way that the rate of erosion is more or less balanced by the formation of new soil from rocks and a state of equilibrium is established between climate, soil, rainfall, land slope and vegetal cover. However, as stated by Jacks and White,[1] "The equilibrium between denudation and soil formation is easily disturbed by the activities of man. Cultivation, deforestation, or the destruction of natural vegetation by over-grazing or other means, unless carried out according to certain immutable conditions imposed by each region, may so accelerate denudation that soil which would normally be washed or blown away in a century, disappears within a year or even within a day. But no human ingenuity can accelerate the soil formation process from lifeless rock to an extent at all comparable to the acceleration of denudation. This man-accelerated denudation is what is now known as soil erosion."

Thus soil erosion is a world-wide malady. It has been one of the most potent causes of the fall of past civilizations and empires whose ruined cities lie buried amid barren wastes that were once fertile lands. The deserts of North Africa, Persia, and Mesopotamia are ghastly reminders of the ravages of soil erosion caused by deforestation, over-grazing and excessive cultivation.

Soil erosion not only removes the good fertile soil from lands, often rendering them incapable of quickly growing even grass and shrubs thus further accelerating sheet erosion and formation of gullies, but at the same time causes other damage which varies from place to place, such as the filling of stream channels, shoaling of navigation channels, depletion of reservoir capacities, obstruction of drainage and burying fertile alluvial lands under sand or gravel.

1. Jacks, G. V. and White R.O., *The Rape of the Earth, World Survey of Erosion* (London, Faber and Faber)

Information regarding the actual state of soil erosion in different countries of the ECAFE region is indeed meagre, but what little information is available goes to show that the problem in this region, with its rapidly increasing population, is even more serious than elsewhere in the world.

In general, the extent and the amount of soil erosion depend mainly upon two factors, namely, the physical and the human. Under physical factor alone natural equilibrium is achieved between the erosive forces of nature such as rain, wind etc. and the resistance of the soil to erosion, which depends on its topography, vegetal cover, soil composition, texture etc. However, there are certain physical forces, such as earthquakes and landslides, which may disturb this natural equilibrium and may cause accelerated erosion. Instances of erosion as a result of non-equilibrium in physical factors are found along the eastern Himalayas in Assam (India), where earthquakes frequently hurl huge masses of soil and rock into the rivers within a few minutes; in Japan, Taiwan and Travancore (India), where saturation of soil on steep mountain slopes after heavy rains causes landslides; and in the Gobi desert in China where strong north-west winds blow away all disintegrated soil particles towards the south.

Erosion in this region, as elsewhere in the world, is essentially a result of human interference which upsets the natural balance. As anti-erosion measures have not yet been practised anywhere to any large extent, with the exception of Japan, the amount and extent of erosion appears to be proportional to the density of the population. Thus, in Indonesia, erosion is quite serious in the densely populated island of Java, but not so in the islands of Sumatra, Kalimantan, and Sulawesi. Generally speaking, countries in the Indo-Chinese Peninsula comprising Burma, Thailand, Laos, Cambodia, Central and South Viet-Nam, and Malaya and also in Borneo, where the density of population is comparatively sparse and where the vast hilly areas are covered by virgin forest and jungles, there is but little erosion. This is evidenced by the fact that the

Table 1

Soil erosion of river basins

River	Station	Drainage area above station (km²)	Mean annual rainfall (mm)	Mean annual run-off (mm)	Run-off coefficient %	Silt content % by weight			Total silt run-off (tons)	Unit silt run-off (tons per km²)	Annual depth of erosion (mm)	% of erosion to run-off	% of erosion to precipitation
						Max.	Mean	Min.					
Damodar (India)	Rhondia	19,9000	1,180	500	42.5		0.285		28,400,000	1,420	0.95	0.190	0.081
Irrawaddy (Burma)	Prome	367,000	1,750	1,430	81.8		0.057		300,000,000	820	0.55	0.038	0.031
Kosi (India)	Chatra	61,600	1,790	988	55.1		0.286		174,000,000	2,820	1.88	0.190	0.105
Mahanadi (India)	Naraj	132,000	1,380	685	49.6		0.067		61,500,000	465	0.31	0.045	0.023
Mekong (Indochina)	Kratie	662,000	1,380	755	54.8	0.05	0.020		100,000,000	151	0.10	0.013	0.0072
Pearl-West (China)	Wuchow	313,000	1,530	789	51.3	0.038	0.0113	0.003	27,946,000	89	0.06	0.008	0.0039
Red (Indochina)	Vietri	113,000	1,500	1,090	73.0	0.70	0.079	0.04	130,000,000	1,150	0.77	0.071	0.051
Yangtze (China)	Chikiang	1,025,000	800	504	63.0	0.56	0.097	0.0015	503,651,000	491	0.33	0.065	0.041
Yellow (China)	Shenhsien	715,184	470	60	12.8	46.14	4.40	0.11	1,890,000,000	2,640	1.76	2.94	0.376
Ching (China)	Changchiashan	56,930	550	29	5.3	54.70			409,150,000	7,190	4.86	16.40	0.867
Colorado (USA)	Grand Canyon, Arizona	356,000		44					261,000,000	735	0.49	1.12	
Colorado (USA)	Lees Ferry, Arizona	279,500		49					240,000,000	858	0.57	1.17	
Colorado (USA)	Cisco, Unah	62,500		98					16,000,000	255	0.17	0.24	
Green (USA)	Green River, Utam	105,600		41.3					28,500,000	270	0.18	0.42	
Little Colorado (USA)		57,100		6					76,800,000	1,340	0.89	14.20	
Rio Grande (USA)	San Marcial, New Mexico	71,900		18					28,000,000	375	0.25	1.27	
San Juan (USA)	Bluff, Utah	62,100		38					51,000,000	916	0.61	1.65	
Mississippi (USA)	Natchez	2,977,000		180					147,300,000	50	0.074	0.041	

maximum sediment content of the Irrawaddy, the Chao Phya, and the Mekong during floods does not exceed one per cent. Exceptions, however, exist in those sparsely populated areas where shifting cultivation is practised.

Soil erosion is moderate in the head water areas of the Red River in North Viet-Nam and China, in the Pearl and the Yangtze basin in South and Central China, in the vast peninsular area south of the Vindhya Range in India, but it is extremely serious in the loessial area of the Yellow River and Hai river basins in North China, on some of the steep hillsides of the Himalayas and in the upper alluvial valley (Shivalik) of the Indus and the Ganga in India, as well as on the very steep mountain slopes in the islands of Japan and Taiwan.

A brief description of the condition of soil erosion in various countries in the region is given below and the sediment load and run-off of some major rivers, both within and outside the region are given in table 1.

CONDITIONS IN VARIOUS COUNTRIES

Burma

Soil erosion in Burma is not very serious except in the hilly catchment of the Chindwin river, in the Kachin and Chin hills and in parts of the Shan States.

The Chindwin river, a tributary of the Irrawaddy, is known to carry large quantities of sediment during the rainy season, while the waters of the Irrawaddy are comparatively clear.

The causes of erosion are, as elsewhere, the destruction of forests, tungya (shifting) cultivation, unscientific downland cultivation and drainage, uncontrolled grazing, grass burning and neglect in the construction and maintenance of paths, roads and railways.

To a very large extent, the disappearance of the forest cover, particularly in the hill tracts, is the direct result of shifting cultivation. The firing of the cut tungya during the dry season destroys all humus and scorches the soil particles, thus rendering the land more vulnerable to soil-wash during rains.

Ceylon

Accelerated erosion in Ceylon is of comparatively recent origin. The two main causes of denudation which is still in progress in Ceylon are: the felling and clearing of forests to make room for plantations without adequate safeguards for erosion and the practice of shifting cultivation called *chena* in Ceylon.

The Committee on Soil Erosion appointed in 1929 held[1] that estate agriculture was responsible for a very large part of the erosion in progress in Ceylon. Tea plantations have caused the greatest denudations due to the clean weeding practised on the gardens located mostly on the steep slopes. The effects of soil denudation on mature rubber plantations are not so apparent as they are on tea plantations, but they are nevertheless considerable. Erosion is, however, greater in areas where the trees are uprooted and burnt for fresh rubber plantations.

In general the treatment of new clearings leaves much to be desired. The steps taken to prevent erosion are either inadequate or adopted too late to be of value.

An enormous amount of damage is done yearly by "patana" fires. The vegetal matter is converted into ash which, in the absence of cultivation, is either washed or blown away, and there is actual erosion during the period between a burn-off and the reproduction of a cover. Moreover, re-establishment of jungle is entirely prevented by successive burnings.

The erosion results in silting of estates and paddy fields, damage to irrigation works, roads and railways, and silting and flooding of rivers.

The Ceylon Government has recently promulgated a comprehensive Soil Conservation Act (No. 25 of 1951) which covers a wide range of subjects and provides wide power, but it needs staff to implement it.

China

In mainland China, very little erosion is found in fields under paddy, whether they are located in the river alluvium of the Yangtze and Pearl river basins, or on the slopes of hills and mountains in the southern and southwestern parts of China where rice fields are carefully terraced. Erosion is considered slight in the North China plain where wheat, cotton and dry crops are cultivated. Slight to moderate erosion prevails over practically every hilly tract and mountainous area wherever it is easily accessible. Excessive erosion takes place in the loessial areas of the Yellow River basin and river basins of north China, which are unique as far as the soils of this region are concerned.

Unlike ordinary soils derived locally from the disintegration of rocks, loess is a wind-blown deposit covering an area of tens of thousands of square kilometres in the northern and north-western parts of China to an average depth of

1. *Report of the Committee on Soil Erosion*, Ceylon Government Press, Colombo, February 1931.

50 m (170 ft). It is very uniform in texture and is highly erodible throughout. From this loess the Yellow River and other rivers in North China derive their large sediment content which, during the flood season, sometimes reaches 50 per cent by weight. The thickness of the soil eroded annually in the basins of this river and of some of its tributaries, as given in table 2, ranges from less than 1 mm to nearly 5 mm. On some steep slopes, a layer of soil 50 mm in thickness is often washed away in a single storm of a few hours duration.

This vast loessial area, including the hilly tracts, becomes absolutely bare after the crops have been reaped and is without any trees, because even grass roots are dug up for fuel. Sheet erosion continuously removes the loess on the surface, gully erosion cuts deep and far back into the sloping land and large blocks of soil material fall into the small creeks and streams. During heavy rains, the flow in the river is nothing but a muddy paste.

Owing perhaps to the fact that the loess deposit, which is about 50 m (170 ft) in depth on the average, is fertile to the last inch of soil and that dry crops are cultivated instead of paddy, the farmers have cared less for measures to conserve the soil as they do elsewhere in China.

Table 2

Station or tributary	Drainage area (km^2)	Recording years	Specific silt yield (t/km^2)			Average depth of erosion (mm)
			Max.	Min.	Average	
Yellow River at Lanchow . . .	316,180	1938, 1941-45	2,740	150	1,110	0.77
Yellow River at Lungmen . . .	515,380	1935-37	2,780	1,079	2,120	1.43
Yellow River at Shenhsien . .	715,184	1934-42	3,690	961	2,640	1.76
Yellow River at Tungkuan . .	712,590	1934-37	2,130	1,091	1,710	1.15
Ching River at Changchiashan .	56,930	1932-45	29,250	1,340	7,190	4.86
Lo River at Chuantou	27,020	1934-45	25,400	127	7,070	4.77
Wei River at Weichiapao . . .	36,000	1943-45	8,560	3,240	5,800	3.92

Density of natural soil 1.48 t/m^3

Of the total area of Taiwai, China, about 64 per cent or 2,278,956 ha (9,116 sq mi) are forest area, out of which 78 per cent or 1,788,116 ha (7,152 sq mi) are classed as productive forests, while the rest or 22 per cent amounting to 490,840 ha (1,963 sq mi) is unproductive.

During the second world war, extensive felling of forests was done, the area cut totalling 127,257 ha (509 sq mi). After the war, although reforestation has continued on a small scale, large areas formerly planted have been lost for lack of proper maintenance. Moreover, large areas of artificial and private forests have been cut indiscriminately for fuel. The total denuded forest area now stands at 236,830 ha (947 sq mi) or nearly 6.6 per cent of the total area of the island where the problem of soil erosion is serious.

Shifting cultivation is prevalent on the steep hill slopes among the aborigines, regardless of ownership. The land is cultivated for a few years and then given up. The bare unprotected land is thus subject to erosion by the heavy rainfall.

The mountains of Taiwan are steep and most of them are of such composition that with a high intensity of rainfall and occasional earthquakes landslides occur quite frequently.

India

There is hardly any province or State in India which is free from the menace of erosion. The kind of erosion and the extent of damage, however, vary in different regions. In the arid parts of the north-west, wind erosion has been responsible for the loss of valuable top soil; sand from the desert of Rajputana is encroaching on the fertile lands towards the east and north-east.

The semi-arid zone covers the foothills comprising the Punjab and Uttar Pradesh. The soils of these foothills being highly erodible, overgrazing and the cutting of the forests have caused tremendous damage in these places. The gradual disappearance of the vegetal cover has resulted in severe sheet erosion and gullying, which often develop in the cattle trails. In 1852 the seasonal torrents in Punjab covered 192 km^2 (75 sq mi). In 1896, the damage extended to

375 km² (145 sq mi) and by 1939 the land destroyed by the torrents amounted to 1,800 km² (695 sq mi).

Perhaps the most striking example of soil erosion in India is to be found along the Yamuna river, where erosion is caused by guillies cutting back into the fertile Gangetic plain. Intensive cultivation is practised in the Indo-Gangetic plain and with the concentrated rainfall that occurs during the three monsoon months considerable volumes of top soil are washed away. In the headwaters of the Ganga and its tributaries soil erosion is of recent growth and has not yet assumed very serious proportions, but with the rapid deforestation that is in progress, conditions may soon become serious.

Landslides due to the removal of forest cover from the hillsides are more prevalent in the central Himalayas. The rivers Kosi, Teesta and Torsa and other Himalayan streams that join the Ganga and the Brahmaputra in this reach are all heavily laden with coarse debris. All these streams are notorious for the sudden changes in their courses as they enter the plains.

In the northeastern regions, as in West Bengal, erosion is due to heavy precipitation and denudation of forest. The heavy silt load of the Damodar floods is principally due to deforestation of its catchment. In Bengal and Assam, river bank erosion and sand deposition are also quite serious.

Along the eastern Himalayas, earthquakes have been one of the most potent factors leading to landslides and soil erosion, which resulted in heavily silt and sand laden waters flowing down the rivers for months and have brought about radical changes in the upper reaches of the Brahmaputra and its tributaries.

In the south and south-east, comprising Madhya Bharat, Bombay, Hyderabad, Madras and Orissa, where the topography is rough and undulating, and where the predominant soils are heavy black and red laterites, generally shallow in depth, erosion is fairly widespread and severe. In addition there is roadside erosion, particularly serious landslides which occur so commonly on the hill roads during the rainy season.

Systematic surveys of the soils of India are being undertaken by the different States and a fair assessment of the extent of deforestation and erosion will be possible when these have been completed.

Indonesia

In Indonesia, the island of Java is the one subject to the menace of soil erosion. The danger of soil erosion in Java, particularly to the coffee plantations and to those lands which were not protected against erosion by terracing, was pointed out by Holle as early as 1866.

Shifting cultivation called "huma" is responsible for large-scale soil depletion in Java. The increase in population during the nineteenth century made larger crops necessary, which it was attempted to obtain from the virgin forest soil with as little labour as possible. The population took to extensive burning to clear the forest; this leaves the surface without any protection against the intense tropical rains which cause the soil to be transported extremely rapidly to rivers and streams.

The damage by soil erosion increased alarmingly owing to the stimulation of cutting and burning of plantations and forest reservations during the second world war. Experiments and observations in Java have indicated conclusively that erosion is dominated by one factor, the degree of bareness of the soil.

Soil erosion is more or less retarded by the rapid regeneration of plant cover in the wet tropics if the soil is left untouched. If, however, the soils are excessively susceptible to erosion, no restoration can be expected. The annual thickness of soil washed away varies from 0.3 mm in volcanic material to 5.0 mm in clay-marly soils.

The effect of increased deforestation, reckless cultural methods and pasturing is shown by the results of experiments on run-off and silt transportation in the Tjiloetoeng basin in Java reported by Van Dyjk and Vogelzang,[1] as shown in table 3.

1. Van Dyjk, J. W. and Vogelzang, W.L.M., "The Influence of Improper Soil Management on Erosion Velocity in the Fjiloetoeng Basin", *Mededelingen van het Algemeen Proefstation voor de Landbouw* No. 71, Buitenzorg. 1948.

Table 3

Items	1911—12	1934—35
Area drained (km²)	620	620
Total rainfall (million m³)	1,791	1,942
Rainfall during wet monsoon (million m³)	1,551	1,823
Total run-off (million m³)	812	863
Run-off during wet monsoon (million m³)	715	779
Run-off coefficient, annual	45	44
Run-off coefficient during wet monsoon	46	43
Maximum silt concentration (g/l)	8.14	21.46
Total silt removed (thousand tons)	821	1,790
Silt removed during wet monsoon (thousands tons)	804	1,765
Silt removed during monsoon (per cent of annual total)	98	99
Total silt removed (t/ha)	13.2	28.9
Yearly erosion (mm) (Volume: weight of soil 1.5 t/m³)	0.9	1.9

Japan

The forest area in Japan amounts to 24.66 million hectares (98,640 sq mi) representing 68 per cent of its total land area. Of this, 20.92 million hectares (83,680 sq mi) is classified as timber land while the remainder is principally waste land. The forest badly damaged by erosion is about 180,000 ha (720 sq mi), and such denuded land is still on the increase.

Because of the steep slopes, the volcanic soil and the heavy rainfall intensity which is often more than 400 mm in a single day, destruction of land by erosion is quite common. In recent years soil erosion has increased, owing to denudation of forest land as a result of over-cutting of trees.

There are few countries which are so steep and so rainy as Japan and the denuded lands are subject to rapid erosion due to the nature of the soil, derived mostly from coarse-grained granite and volcanic ash and lava.

During the second world war and even before it, all principles of timber management and protection of forest were put aside in favour of increased production, and extensive clear cutting of forest areas was resorted to in order to meet the needs for timber. As a result, flood and erosion damages developed to such a stage that it became a national emergency. The soil conservation measures comprise mainly "Sabu-works" which are intended to protect headwater areas from damage. These are generally divided into two parts, one to protect mountain side from erosion, and the other to check the transport of debris from the headwater areas. The former are generally under the charge of the Forest Agency of the Ministry of Agriculture and Forestry, and the latter under the River Bureau of the Ministry of Construction.

THE PROBLEM

In view of the inadequacy and meagreness of the information available regarding the state of soil erosion in almost every country of the ECAFE region, the most pressing problem is a quick survey of the areas subject to severe erosion. Aerial surveys have often been suggested for this work, but they are considered expensive. It is, therefore, necessary to develop cheap and quick methods of survey for each country according to its requirements. The focal points of erosion could be located by observing the silt load in rivers and their tributaries and smaller streams, which should enable the source of the sediment and, therefore, the area of severe erosion to be located. The problem is colossal in almost every country, and the necessity of selecting specially bad areas for treatment so as to get quick results at minimum cost is apparent. Not only is it necessary to locate the areas of sheet erosion, but also of gully erosion, stream channel erosion and other sources of sediment so as to frame unified plans of soil conservation and sediment control for entire basins.

Chapter II

TRANSPORTATION OF SEDIMENT

SEDIMENT PROPERTIES

The picking-up, transportation and deposition of sediment depends upon the physical properties of sediment as well as the hydraulic characteristics of flow. Sediment in motion consists usually of particles of different sizes, shapes and specific gravities and for convenience of analysis, certain parameters, typifying the aggregate behaviour of the group of sediment particles under consideration, have to be chosen. Before doing so, a description of the properties of a single particle will help to clarify the problem.

Properties of individual particles

Of the various properties of sediment, the sediment size is perhaps the one most commonly used to designate its properties. Strictly speaking, it is the size of sediment and its packing that directly affect the roughness of the bed; but it is the weight, the form, and the terminal fall velocity of the sediment particle in water that govern directly the movement of sediment. However, all these properties are related to and can be easily expressed in terms of the size of the sediment. The size of sediment, therefore, is still considered to be sufficient to describe its properties.

Sediment particles larger than 0.06 mm in diameter can be analyzed by sieves, and the opening of the sieve is used to designate the particle size. For particles of diameter smaller than 0.06 mm which is the practical lower limit for sieve analysis, size analysis is usually done by determining the terminal fall velocity of the particle in water. The diameter of a quartz sphere having the same terminal fall velocity as that of the particle under observation in the same fluid is taken as the diameter of the particle. The terminal fall velocity of quartz particles in relation to the diameter is given in figure 1.

The specific gravity of sediment generally varies within a narrow range, between 2.50 and 2.70. Since practically all stream-borne sediment has its origin in the disintegration or decomposition of rock material, essentially all constituents of the parent rock are found in sediments. The finest materials in suspension are primarily clay minerals derived from the decomposition of feldspar and mica. The composition gradually changes towards the relatively inert silica in the silt range (0.004 to 0.06 mm). Sands (0.06 to 2.0 mm) are almost pure silica or quartz. Sediment particles above the size usually consist of recognizable fragments of parent rock or all the original rock components.

Group of sediment particles

In practice, sediments consist of innumerable particles differing in size, shape, specific gravity and terminal fall velocity. It is desirable to find out some parameters that can represent the characteristics of the group of sediment particles as a whole. Generally, the particle size or the terminal fall velocity is used as the parameter.

A sample of sediment is usually divided into certain class interval according to size or terminal fall velocity and the percentage of the total in each class (according to number, volume or usually weight) is ascertained. By plotting the percentage against the diameter or terminal fall velocity, with the latter usually in geometric progression, one obtains a curve called frequency distribution of the sample. The important parameters defining the composition of the sample are the mean diameter, the standard deviation from the mean, and the skewness.

The mean diameter is the most important parameter among the three. The standard deviation[1] from the mean gives the dispersion or scatter about the mean. The skewness is perhaps of minor importance. Curves A and B in figure 2 show two unskew distribution curves having

1. In the frequency curve shown in figure, 3 the mean diameter is given by the expression

$$\bar{d} = \frac{\Sigma\ (\text{moment of area } ydx)}{\Sigma\ (\text{area enclosed in the curve } y = f(x))}$$

$$= \frac{\int_0^{x_1} (ydx)\ x}{\int_0^{x_1} ydx} = \frac{\int_0^{x_1} xydx}{\int_0^{x_1} ydx}$$

The standard deviation is defined as

$$\Delta = \sqrt{\frac{\Sigma (d - \bar{d})^2}{n - 1}}$$

where d is the diameter of each class interval, $\bar{d}$ the mean diameter and N the number of class intervals.

different means but the same standard deviations; curves B and C show two unskew distributions having the same mean but different standard deviations; while curve D shows a skew distribution curve.

A cumulative curve is usually used in place of a distribution curve. The cumulative curve is obtained by plotting the percentage of sediments (by number or volume or usually by weight) smaller or larger than the corresponding size of sediment, which is in fact a summation of the distribution curve. The abscissa indicating particle size or terminal fall velocity is plotted on logarithmic or natural scale while the percentage is plotted either on natural or probability scale. Figure 4 shows a cumulative curve plotted on a natural scale, with diameter as a parameter for the size.

Instead of using the mean diameter[1] and standard deviation derived from the distribution curve, many research workers have introduced other characteristic diameters from the cumulative curve to describe the characteristic of the sample. These characteristic diameters are usually the diameter corresponding to a certain percentage in the cumulative curve such as $d_{20\%}$, $d_{50\%}$ or d median, and $d_{80\%}$, meaning that 20, 50 or 80% of the particles are smaller or larger than this size.

George H. Otto[2] carried out a thorough study in the Soil Conservation Service, Co-operative Laboratory, California Institute of Technology, Pasadena, California on the mechanical analysis of sediments by comparison with other analyses and outlined a general procedure for systematic interpretation of mechanical analysis which he found to be of great advantage.

MECHANISM OF TRANSPORTATION

It is commonly observed that flowing water can transport sediment. Thus the muddy appearance of river water during floods is a clear indication of sediment being carried by the water in suspension and the clicking sound of pebbles striking each other, which is distinctly heard while crossing a swiftly running stream, indicates the rolling of sediment over the river bed. However, it is only experiments conducted in glass flumes in laboratories that make possible the actual observation of the movement of sediment and clarify some of the details of this phenomenon.

A study of the movement of non-cohesive material along the bed of a straight uniform flume subjected to the action of flowing water will show that with a certain depth of flow, when the velocity of flow increases to a certain value, some particles of sand, presumably those protruding out of the bed, commence intermittent movement by rolling and sliding along the bed. With further increase of the velocity of flow (accompanied by an increase of discharge for the same depth), particles rolling and sliding along the bed increase in number. Depending upon the physical characteristics of the bed material (size, composition, shape, etc.) and with the increase in the rate of bed material movement, the original level bed gradually assumes an undulating form consisting of ripples (or bars) which resemble sand dunes on a dry sandy beach with a gentle slope on the windward side and a steep slope on the leeward side. Sand particles roll upwards along the gentle slope and then suddenly jump clear over the crest of each ripple or bar. The larger ones just drop down over the crest and deposit right in front of the downstream slope of the ripple while the smaller ones are carried over and deposited further downstream, giving the appearance of simultaneous movement of a series of ripples towards the downstream side. The hopping of particles, known as saltation, is intensified as the velocity of flow increases. With further increase of velocity, particles swept away from the bed are kept in suspension by the upward component of the turbulent velocity of flow and are thus carried downstream without being deposited on the bed. This action is often intensified by turbulent eddies which have vertical axes. These eddies uproot the bed material and throw it in suspension to be carried by the flow.[3]

The above description indicates the various phases of sediment movement for a certain kind of sediment material under variable hydraulic conditions of flow. If the hydraulic conditions of flow remain constant while the composition of the sediment varies, as in a natural stream carrying varying sizes of sediment, it can be shown that for a certain condition of flow, the smallest particles such as clay and silt will be carried in suspension, larger particles such as sand will be in saltation while pebbles and gravel will tend to roll and slide over the bed.

Thus, depending upon the condition of flow and on the composition of the sediment, the different phases of sediment transportation change gradually and continuously from one to the other. There can be no sharp line of demarcation between the different phases. How-

1. The mean diameter can also be obtained from the cumulative curve as follows: As already stated the cumulative curve is obtained by plotting the intergral of the distribution curve against diameter of particles. Referring to Figures 4 and 5

$$J = \int_o^x ydx$$

$$dJ = ydx$$

The mean diameter is given by the expression

$$\bar{d} = \frac{\int_o^{x_1} xydx}{\int_o^{x_1} ydx} = \frac{\int_o^{100} xdJ}{\int_o^{100} dJ}$$

which is the mean abscissa of curve oa with respect to ob or area oabo divided by ob, (figure 5).

2. Otto, George H.: "A modified Logarithmic Probability Graph for the Interpretation of Mechanical Analysis of Sediments". *Journal of Sedimentary Petrology*, Vol. 9, No. 2, August 1939.
3. Bhandari, N.N., *A study of the Turbulence created by the Discharging of a Jet into a Mass of the same Fluid*, University of London, 1941.

ever, for convenience of analysis of this complicated problem, sediment load is usually divided into two categories, bed load and suspended load. Bed load is also sub-divided into contact load and saltation load. Contact load denotes material that is rolling or sliding along the bed whilst the saltation load is the sediment bouncing and hopping along the bed. The suspended load is that what is kept in suspension in the fluid.

Very often another term "wash load" is used to designates the portion of the rather fine suspended particles which do not originate from the river bed but from the headwater area, owing to soil erosion. However as far as the particle size of washload is concerned it is just a comparative term. For example sand, an essential element in the composition of river bed for ordinary alluvial rivers, may be considered as wash load for steep rivers with their bed composed of and also carrying gravel and boulders.

From the practical point of view, sediment movement in alluvial streams and in canals located in alluvial soil is of special significance, where river bed materials are derived entirely from the deposits of flow and are frequently picked up again by the current. It is mainly the sediment transportation in the transitional region between bed and suspended load movement, where, in fact, satisfactory knowledge is particularly lacking.

SEDIMENT TRANSPORTATION IN CLOSED CONDUITS

The transportation of sediment in a closed conduit differs from that in an open channel in two respects: firstly, the absence of a free surface and secondly, the presence of a fixed boundary[1] in contrast to the flow in rivers and canals where the boundary is composed of erodible material. However, if suspended load dominates the flow in an open channel such as the conveyance of heavily silt-laden flow under high velocity in a lined canal so that the boundary conditions play a less important role, the results of pipe tests within certain limits may be applicable and may throw some light on the problem. Conducting investigations in a closed conduit has many advantages because various factors such as velocity of flow and concentration of sediment etc. can be easily controlled and the loss of head can be easily measured which is rather difficult in an open channel.

The results of experiments on the transportation of gravel, sand, clay and sawdust by Siefgried, Gregory, Durand[2] are shown in figure 6. The quantity of sediment transported or the concentration of sediment is expressed as a percentage of volume of sediment to volume of suspended flow. With loss of head per unit length of pipe (expressed in terms of height of mixtures so as to take into account the density of the fluid at different concentrations) as ordinates and mean discharge or velocity of flow as abscissa, lines of equal concentration were plotted in the diagramme. Line of zero concentration, or clear water flow, is also plotted in the diagramme for comparison.

The curves of equal concentration (c curves) appear to have a point of minimum value with respect to loss of head. According to Durand, this point of minimum value corresponds to the stage of sediment flow at which no deposition occurs at the bottom of the pipe. The parts of c curves lying on the left hand side of their minimum points show an increase of lose of head for a decrease of velocity of flow. This is attributed to the fact that, owing to the decrease of velocity of flow, the lower portion of the pipe is filled up with sediment. The reduction of the actual cross-section area in the pipe accounts for the increase of loss of head.

A study of the portions of the curves situated on the right hand side of their minimum points will, however, show that the loss of head, expressed in terms of height of mixture, increases with the increase of concentration. For the same concentration, the loss of head also increases with the increase of particle size of sediment. When the velocity of flow (more correctly expressed by the Reynold's Number) is large, the various c curves approach the curve of clear water. At this stage, the mixing of the sediment with water is so violent that the sediment flow may be considered as a "true suspension." At the points where the c curves approach the curve for clear water flow, a lower velocity (or Reynolds' Number) is required for lower concentration, and for the same concentration, a lower Reynold's Number is required for smaller size of sediment. Most interesting are the results of Gregory's tests with "clayey mud" in which a few c curves fall below the curve of clear water at certain range of velocities.[1] This means that the loss of head or friction factor[2] for the flow with certain concentrations is lower than that for clear water flow. Mention may here be made of the experiments conducted by Vanoni[3] on sediment flow in open channels. Vanoni found in his experiments that for an

1. At high velocoity of flow when sediment is moving or sliding over the wall of a pipe.
2. Durand, R., "Transport hydraulique de gravier et galets en conduite", in *La Houille Blanche*, 1951, No. B, P.609, and The Hydraulic Transportation of the Coal and Solid Material in Pipes", Paper presented at the London Colloquium of the National Coal Board, November 1952. See also Raynaud, J.P., "Study of Saturated Rivers by means of a Graphical Representation", 4th Meeting of the International Association for Hydraulic Research, Bombay, 1951, R 4 Q 3.

1. There is no reason why the various c curves should cross each other. Attention is also drawn to the fact if the loss of head is expressed in terms of head clear water rather than head (height) of mixture the c curves will be always above the curve of clear water.
2. Frictionfactor ε is defined as $\varepsilon = H_\varepsilon \Big/ \left(\frac{1}{R} \frac{V^2}{2g} \right)$
3. Vanoni, Vito A., "Transportation of Suspended Sediment by Water", in *Transactions of the A.S.C.E.*, 1946, p. 101.

average suspended load of 1.2 g/l (with mean diameter of sediment range from 0.10 to 0.16 mm), the friction factor ε is reduced by as much as 20 per cent. He explained that this was due to the damping of turbulence by sediment in suspension. Durand also made the observation that "fine solids seem to reduce the head losses considerably." It is of importance to note that this phenomenon is observed only in Gregory's tests in which clayey mud was used but not in other experiments in which sediments of larger sizes such as sand and gravel were tested.

As a result of a very extensive series of experiments using pipes of diameters varying from 40 to 480 mm, diameter of sediment from 0.2 to 25 mm and concentration from 50 to 600 g/l, Durand[1] determined the relation between the loss of head or friction gradient in relation to discharge, concentration and particle size. The equation given by Durand is derived for the condition of sediment flow above the deposition limit (i.e. for values lying on the right of the minimum points of the c curves). The results obtained for sand, gravel and pebbles of specific gravity 2.65 are plotted in a non-dimensional form in figure 7. The equation of the curve is given by the expression

$$\frac{S_e - S}{S\ c} = \text{Const.} \left(\frac{\sqrt{gD}}{V}\right)^3 \left(\frac{1}{\sqrt{C_d}}\right)^{3/2} \quad (1)$$

$$\text{or } S_e = \varepsilon \frac{V^2}{gD} + \text{Const. } \varepsilon \frac{\sqrt{gD}}{V} \cdot \frac{c}{(\sqrt{C_d})^{3/2}} \quad (2)$$

where S_e = friction slope of sediment flow in pipes expressed in height of clear water per unit length of pipe

S = friction slope of clear water flow at corresponding V and D

c = concentration of sediment expressed in volume of sediment transported per unit discharge of mixture

g = gravitational constant

D = diameter of pipe

V = mean velocity of flow

C_d = drag coefficient of sediment = $\left(\frac{4\ gd}{3\ w^2}\right) \frac{\rho' - \rho}{\rho}$

ε = friction factor of clear water flow = $2g\ SD/V^2$

w = terminal fall velocity of sediment in water

d = diameter of sediment

ρ' = density of sediment

ρ = density of water

The above equation is valid for sediment of density equal to 2.65. For sediments of other densities, Durand suggested that the expression

$$\left(\frac{\sqrt{gD}}{V}\right)^3 \left(\frac{1}{\sqrt{C_d}}\right)^{3/2} \quad \text{be replaced by}$$

$$\left[\frac{\sqrt{gD\frac{\rho' - \rho}{\rho}}}{V}\right]^3 \left(\frac{1}{\sqrt{C_d}}\right)^{3/2}$$

The sets of diagrams shown in figure 6 and the formula derived by Durand are of great practical utility for designing pipe-lines of dredgers, as well as pipe-lines for conveyance of coal and paper pulp, etc. Referring to curves shown in figure 6, the velocity of pipe should be selected from the minimum value of c curves.

Durand[2] also presented a set of generalized diagrams indicating the possible phases of movement of sediments of various sizes as shown in figure 8. He divided the sediment into three classes: (1) those of diameter d with a falling velocity w in water following Stokes' Law i.e. $w \propto d^2$; (2) the larger particles for which $w \propto \sqrt{d}$; and (3) particles of sizes lying in between those two. The probable regions of various phases of sediment movement are also indicated in the diagrams.

Experiments about sediment flow in pipes certainly provide a clear picture of the various phases of sediment transportation and cover a wide range of variables which can only be reproduced in open channel with considerable difficulty. Applying the experimental results in pipe to open channel, the non-deposition régime can be considered similar to the sediment transportation in a lined channel when the velocity is sufficiently high to keep channel clear from any deposits. As the natural river bed or canal bed is usually composed of erodible material over which sediment is transported, it is precisely the deposition régime in pipe flow that bears similarity to open channel flow and requires further investigation.

An important observation may be made by comparing the loss of head of a mixture with that of clear water. In can be seen that the the loss of head of a mixture, expressed in terms of loss of head to clear water, is much larger when sediment is moving as bed load than in the case of suspended load. This shows that sediment moving near the bed has far greater influence on the hydraulic factors of flow (such as loss of head, friction factor, etc.) than suspended load. This is important in dealing with problems of river hydraulics.

1. Durand R.: "The Hydraulic Transportation of Coal and Solid Material in Pipes", Paper presented at the London Colloquium of the National Coal Board, November 1952.

2. Durand, R.: "Transport hydraulique de gravier et galets en conduite", in *La Houille Blanche*, 1951 No. B, p. 609.

SEDIMENT TRANSPORTATION IN OPEN CHANNELS

Bed-load transportation

Two questions enter into the problem of bed-load transportation; namely the law governing the commencement of bed material movement, and the law relating to the rate of bed-load transportation. Starting with the bed material at rest and with the increase of the force of the current acting on it the movement of sediment takes place rather gradually. Particles protruding from the bed begin to move first and the number of particles in motion gradually increases. The critical stage of movement is therefore defined differently by different authors as initial movement, general movement etc. A more reasonable definition[1] is to plot the rates of transportation against tractive force and to extrapolate the rate of transportation to zero. The corresponding tractive force will then give the critical stage of bed load movement.

(*a*) *The critical tractive force*

For uniform flow, the force of running water exerted on the bed of a channel, known as tractive force τ, is equal to γRS, where γ is the specific weight of water, R the hydraulic mean radius and S the slope. Krey[2] reasoned that the resistance of sediment to motion is proportional to the diameter of sediment d, specific weight of sediment in water $\gamma\ (s_s - 1)$, where γ is the specific weight of water and s_s the specific gravity of sediment. For the critical tractive force or the tractive force at the critical stage of bed load movement, Krey suggested the following dimensionally homogeneous equation in which C is a dimensionless coefficient having a value obtained from the expression:

$$\tau_c = C\gamma\ (s_s - 1)\, d$$

or $$\frac{\tau_c}{\gamma\ (s_s - 1)\, d} = \text{Const.} = 0.076$$

Shields[3] showed that $\dfrac{\tau_c}{\gamma\ (s_s - 1)\, d}$ is not a constant but a variable. Assuming that the force exerted by the flow upon a sediment particle could be expressed in terms of the usual drag relationship i.e.

$$F = CA\gamma \frac{V^2}{2g} = \phi_1\ (\alpha_1, \frac{Vd}{\nu})\ \gamma\ \frac{\pi d^2}{4}\quad \frac{V^2}{2g} \qquad (3)$$

in which C is the drag coefficient which is usually a function of Reynold's number and shape of the particle, α_1 is a shape factor for the particles, and V the velocity of flow at an elevation Z proportional to the diameter d of the particle (i.e. $Z = \alpha_2 d$). The velocity V can be expressed by the following formula based on the results of experiments on resistance of artifically roughened plates[4] i.e.

$$v = \sqrt{\frac{\tau g}{\gamma}}\ \phi_2 \left(\alpha_2,\ \frac{d\sqrt{\frac{\tau g}{\gamma}}}{\nu} \right) \qquad (4)$$

in which τ is the boundary shear or tractive force. Combining equations (3) and (4) Shields obtained

$$F = \tau\, d^2\, \phi_3 \left(\alpha_1,\ \alpha_2,\ \frac{d\sqrt{\frac{\tau g}{\gamma}}}{\nu} \right) \qquad (5)$$

He further assumed that the resistance of particles to motion depends upon the roughness of the bed and the immersed weight of the particle, i.e.,

$$r = \alpha_3\ (s_s - 1)\ \gamma\ \left(\frac{\pi d}{6}\right)^3 \qquad (6)$$

Equating equations (5) and (6), an equation is obtained in the following form:

$$\frac{\tau}{\gamma\ (s_s - 1)\, d} = \phi_4 \left(\alpha_1,\ \alpha_2,\ \alpha_3,\ \frac{d\sqrt{\frac{\tau g}{\gamma}}}{\nu} \right) \qquad (7)$$

In the case of a level bed comprising particles of uniform size, the several coefficients involving characteristics of sediment and bed can be grouped into one constant, and the resulting function takes the simple form

$$\frac{\tau}{\gamma\ (s_s - 1)\, d} = \phi \left(\frac{d\sqrt{\frac{\tau g}{\gamma}}}{\nu} \right) \qquad (8)$$

Figure 9 shows the form of the function as determined by Shields for a considerable range of each of the different variables. As pointed out by Rouse,[5] Shields' experiments were limited to particles of uniform size and that for non-uniform bed material the curve may take different form.

1. Lane, E. W., "Progress Report on Studies on Design of Stable Channels by the Bureau of Reclamation", *Proceeding Separates of the American Society of Civil Engineers*, No. 280, Sepptember 1953.
2. Krey, H., *Wiederstand von Sand Koernern und Kugeln bei der Bewegung des Wassers*, E.S. Mittler and Sohn, 1921. (See *A. Schoklitsch, Handbuch des Wassersbaues* 2nd edition, Springer, Wien. 1950).
3. Shields, A., "Anwendung der Aehnlichkeitsmechanik und der Turbulenzforschung auf die Geschiebebewegung." *Mitt. Preuss. Versuchsanstalt f. Wasserbau u. Schiffbau, Berlin*, Heft 26, 1936.
4. Rouse, Hunter, "*Engineering Hydraulice* (John Wiley and Sons, New York, 1950), p. 108.
5. *Ibid.*, p. 791.

Very important in the application of such experimental results is the range within which the formula is applicable, as such functions, rightly presented as they were, can not be extrapolated without introducing appreciable error.

For all practical purposes the function ϕ may be assumed to be constant. With the specific gravity γ for gravel, sand and silt taken as 2.65, the critical tractive force would then vary directly with the first power of the diameter d of sediment particles.[1] A plot of the results of various authors for τ against d, as prepared by Lane,[2] is given in figure 10. Scattering of the points may be attributed to the difference in defining the "starting of bed load movement", as well as the difference in shape and composition of sediments used, as the parameter "mean diameter" is usually not sufficient to define the character of the sediment.

For practical use Lane suggested a set of τ-d relations which are given in figure 11. It will be noted that values tentatively recommended by Lane for clear water in fine, non-cohesive material are very much higher than those used in a laboratory but usually have a little binding material which greatly increases the resistance to motion. A higher value of critical force is also given by flow carrying fine colloidal sediment on account of the binding effect exerted by it on the non-cohesive bed material.

The expression $\tau = \gamma RS = \gamma \frac{A}{P} S$ gives only the mean tractive force over the perimeter of a cross-section without defining the distribution of the tractive force along the perimeter. For sections with large ratio of width W to depth D, the hydraulic radius R approaches the mean depth $D_m = A/W$ and therefore $\tau = \gamma\ D_m\ S$. This again gives the mean tractive force of the section and cannot be interpreted to mean that the distribution of tractive force is proportional to the depth within a cross-section.

The actual distribution of tractive force will depend upon the distribution of velocity gradients within the cross-section following the law $\tau = \mu \frac{dV}{dD}$. Parandtl[3] has given the following expression for the turbulent shear stress at any point in a fluid moving past a solid wall.

$$\sqrt{\frac{\tau}{\rho}} = l \frac{dV}{dy} \qquad (9)$$

where

τ = the sheering stress at the point
ρ = the density of the fluid
V = the velocity at the point
y = the distance of the point from the wall, and
l = the so-called mixing length of the momentum exchange.

As a consequence of his principle of similarity of turbulance Karman derived an expression for the mixing length l and finally obtained the equation[4]

$$\frac{V}{V_*} = \frac{1}{K} \log_{10} \left(\frac{y}{y_o}\right) \qquad (10)$$

in which

$V_* = l \frac{dV}{dy} \left(\frac{\tau_o}{\tau}\right)^{\frac{1}{2}}$
τ_o = shear in the fluid at the wall
K = universal Constant characterizing the turbulance
y_o = constant of integration.

This is Karman's law of velocity distribution in the neighbourhood of a solid wall. The derivation is made for small values of y. Experience, on the other hand, shows that equation (10) is sufficiently accurate also for large values of y. Resorting to experience and using the method of dimensional reasoning as a guide Keulegan obtained the universal law of velocity distribution for rough surfaces given by

$$\frac{V}{V_*} = a_r + \frac{2.30}{K} \log_{10} \left(\frac{yV_*}{\nu}\right) \qquad (11)$$

where a_r is a function of the Reynolds number $\frac{k V_*}{\nu}$ and is given by the expression

$$f\left(\frac{k V}{\nu}\right) = e^{-K a_r} \qquad (12)$$

where k is the height of roughness. The function f is known completely only for roughness produced by closely packed grains of sand, as determined by Mikuradse[5] by experiments on pipes of circular cross-section. Nikuradse found that when $\frac{k V_*}{\nu}$ is less than about 3.3, a_r was independent of V_* and has the value 5.5, and the surface behaves as if it were smooth. When $\frac{k V_*}{\nu}$ is greater than about 67, a_r is given by the expression

1. It may be pointed out that, for sediment composed of Particles of various sizes, different "representative" diameters or sizes have been used by different authors, such as mean diameter, weighted mean diameter or a diameter corresponding to certain percentage of the particle size summation curve.
2. Lane, E. W., *op. cit.*
3. Prandtl, L.: "Ueber die ausgebildete Turbulenz," *Proceedings of the 2nd International Congress on Applied Mechanics*, Zurich, 1926, p. 62.
4. Keulegan, Garbis H.: "Laws of Turbulent Flow in Open Channels". United States Department of Commerce, National Bureau of Standards Research Paper RP 1151. *Journal of Research of the National Bureau of Standards*, Vol. 21, December 1938.
5. Nikuradse, J., "Stromungsgestze in rauhen Rohren". *Ver. Deut. Ing., Forschungsheft* 361 (1933).

Table 4

Relative boundary shear for channels of unit depth D in terms of γDS

Method		Membrane analogy												Analytical	Finite difference		
Shape of channel		Trapezoidal						V-notch					Rectangular	Rectangular	Rectangular		
Side slope as		2:1	2:1	2:1	3/2: 1	3/2: 1	3/2: 1	2:1	3/2: 1	1:1	2/3: 1	1/2: 1	0:1	0:1	0:1	0:1	0:1
Bottom width b		2	4	8	2	4	8	0	0	0	0	0	2	2	2	4	1
Side boundary points, vertical distance above bottom	1.0	0	0	0	0	0	0	0	0	0	0	0	0.680*	0.686*	0.686*	0.744*	0.468*
	0.9	0.130	0.130	0.120	0.160	0.190	0.160	0.130	0.160	0.230	0.270	0.275	—	.677	.676	.740	—
	0.8	.250	.260	.340	.320	.330	.300	.260	.290	.350	.350	.320	.660	.654	.664	.724	.460
	0.7	.380	.380	.360	.450	.450	.420	.380	.400	.440	.375*	.325*	—	.625	.636	.700	—
	0.6	.500	.490	.470	.550	.570	.530	.470	.480	.470	.370	.305	.620	.597	.610	.664	.435
	0.5	.600	.590	.580	.620	.650	.610	.550	.530	.480*	.350	.275	—	.566	.560	.610	.415
	0.4	.680	.680	.660	.690	.710	.690	.610	.560	.470	.320	.235	.500	.523	.510	.550	.385
	0.3	.730	.740	.730	.730	.750*	.740	.650*	.565*	.440	.270	.190	—	.449	.436	.476	.342
	0.2	.760*	.770*	.770*	.735*	.740	.760*	.640	.520	.350	.190	.130	.320	.335	.340	.360	.278
	0.15	.760	.765	.760	.720	.710	.710	.600	.480	—	—	—	—	—	—	—	.235
	0.1	.740	.750	.720	.670	.670	.630	.550	.400	.230	.110	.065	.180	.180	.200	.220	.180
	0.05	.670	.660	.640	.530	.590	.520	.440	.290	.130	—	—	.100	—	—	—	.110
	0.025	—	—	—	—	—	—	—	—	.070	—	—	.060	—	—	—	—
	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bottom boundary points, horizontal distance from outside edge toward center	0.025	—	—	—	—	—	—	—	—	—	—	—	.060	—	—	—	—
	0.05	—	—	—	—	—	—	—	—	—	—	—	.100	—	—	—	.110
	0.1	.700	.730	.720	.660	.640	.660	—	—	—	—	—	.180	.211	.200	.220	.180
	0.2	.770	.800	.770	.730	.740	.730	—	—	—	—	—	.320	.345	.340	.360	.275
	0.3	.800	.840	.810	.780	.790	.780	—	—	—	—	—	—	.436	.436	.470	.332
	0.4	.830	.870	.840	.810	.830	.820	—	—	—	—	—	.500	.507	.510	.556	.362
	0.5	—	—	—	—	—	—	—	—	—	—	—	—	.563	.560	.624	.372*
	0.6	.870	—	—	.850	—	—	—	—	—	—	—	.620	.605	.610	.684	—
	0.8	—	.930	.920	—	.910	.910	—	—	—	—	—	—	—	—	.770	—
	1.0	.890*	—	—	.890*	—	—	—	—	—	—	—	.660	.658	.664	.830	—
	1.2	—	.950	—	—	.950	—	—	—	—	—	—	.680*	675*	.686*	.870	—
	1.6	—	.960	.970	—	.965	.960	—	—	—	—	—	—	—	—	.920	—
	2.0	—	.970*	—	—	.970*	—	—	—	—	—	—	—	—	—	.936	—
	2.4	—	—	.980	—	—	.980	—	—	—	—	—	—	—	—	—	—
	4.0	—	—	.990*	—	—	.990*	—	—	—	—	—	—	—	—	—	—

* Maximum values of bottom and side are marked with *.

$$a_r = 8.5 - 5.75 \log_{10} \left(\frac{k\,V}{\nu}\right) \qquad (13)$$

which on substitution in equation (11) gives

$$\frac{V_*}{V} = 8.5 + 5.75 \log_{10} \left(\frac{y}{k}\right) \qquad (14)$$

which may be called Keulegan's law of velocity distribution in the vicinity of a surface covered with closely packed sand grains. Since the vicinity of the fluid does not enter equation (14), the equation applies to flow in the region where hydraulic resistance varies with the square of the mean velocity. Introducing the concept of equivalent roughness Keulegan further simplified equation (14) and made it independent of the shape of the channel. Thus

$$\frac{V}{V^*} = 6.25 + 5.75 \log_{10} \left(\frac{R}{k_s}\right) \qquad (15)$$

in which R is the hydraulic radium and k_s is the roughness independent of the size, distribution and packing of the grains of sand but is equivalent to the particular roughness k.

The problem of distribution of τ in trapezoidal and rectangular cross-sections, in terms of γDS, has been worked out by Olsen and Florey[1] by membrane analogy, analytical and finite difference methods and their results are given in table 4.

Another problem of practical importance is the effect of meandering on the critical tractive force. It is a well known fact that in a winding channel the normal distribution of velocity is distorted and the point of maximum velocity shifts towards the concave bank.

The development of secondary or cross currents greatly increases the tractive force of a stream over that indicated by the equation which holds good only for straight channels.

1. Olsen, R. G. and Florey: "Sedimentation Studies in Open Channels-Boundary Shear and Velocity Distribution by Membrane Analogy, Analytical and Finite Difference Methods", US Bureau of Reclamation *Laboratory Report* No. SP34.

Lane therefore tentatively suggested a reduction of critical tractive force for different degrees of "Sinuosity" of channel and his proposed values are given in table 5. This table is intended to be used for the design of irrigation channels.

Table 5

Critical tractive force—sinuous cands

(after E. W. Lane)

Degree of sinuosity	Percentage critical tractive force required to initiate bed load movement as compared with straight canals	Corresponding percentage of mean velocity
Straight canal	100	100
Slightly sinuous canals	90	95
Moderate sinuous canals ..	75	81
Very sinuous canals	60	78

(*b*) *Rate of transportation*

When the tractive force of a stream exceeds the critical value, certain quantity of sediment is set in motion. The first equation on the rate of sediment transportation was given by Du Boys in which he assumed that the rate of sediment transportation was proportional to the excess of prevailing tractive force over the critical value required to initiate movement.

Thus:

$$q_s = C_s\,\tau\ \ (\tau - \tau_c) \qquad (16)$$

where q_s is the rate of transportation in volume of sediment per unit width, C_s is a coefficient dependent on the character of sediment, and τ and τ_c are the prevailing and the critical tractive force respectively. Straub[2] summarized the results of various investigators and gave the value of C_s and τ_c for various sizes of sediment with specific gravity 2.65 as shown in table 6.

2. Straub, L.G., House Document 238, 73rd Congress, 2nd Session (US Government Printing Office, Washington D.C., 1935) p. 1135.

Table 6

q_s	Size of sediment (mm)	1/8	1/4	1/2	1	2	3
ft³/s/ft	τ_c (lb/sq ft)	0.016	0.017	0.022	0.032	0.051	0.09
m³/s/m	τ_c (kg/m²)	0.078	0.083	0.108	0.156	0.250	0.44
ft³/s/sft	C_s (ft⁶/lb² sec)	8.81	0.48	0.29	0.17	0.10	0.06
m³/s/m	C_s (m⁶/kg² sec)	0.0032	0.0019	0.0011	0.00067	0.00039	0.00023

Table 7

Investigator	Formula	Sediment	Equation No.
US Waterway Exp. Station	$q_s = \frac{C}{N} (\tau - \tau_c)^m$	sand mixture	(17)
Chang, Y. L.	$q_s = CN\tau \ (\tau - \tau_c)$	uniform sand	(18)
O'Brien	$q_s = C \left(\frac{V}{R^{1/3}} \right)^m$	sand mixture	(19)
McDougull	$q_s = CS^m \ (q - q_c)$	sand mixture	(20)
Meyer Peter	$q_s = (C_1 S q^{2/3} - C_2 d)^{3/2}$	uniform sand large size	(21)

Various authors propose many empirical formulae which are given in table 7

in which q_s = rate of sediment transportation in volume per sec. per unit width
q = discharge in volume per sec per unit width
q_c = critical discharge at which sediment movement commences
S = slope or energy gradient
τ = tractive force
τ_c = critical tractive force
C, C_1, C_2 = empirical constant with dimensions
m = empirical exponent, and
N = Manning's roughness coefficient

The fact that there are so many empirical formulae, each of a different form, is rather confusing. Johnson[1] compared the formulae listed above by plotting the same data according to the different formulae. By means of statistical analysis of the various plotted graphs, Johnson found that the goodness of fit for all the plotted data was about the same and therefore he concluded that the choice of equation could be made on the basis of convenience in measuring the variables appearing in the formulae. It should be remembered that all these formulae are valid only for interpolation within the ranges covered by the experiments or measurements on the basis of which they were derived. The maximum q_s covered by the various authors mentioned above was of the order of 0.030 m^3/sec per metre width of channel or 0.3 cu ft/sec per foot.

A dimensionally homogeneous equation for sediment of uniform size, also taking into account the effect of specific gravity of sediment, was proposed by Shields as below:

$$\frac{q_s \ s_s}{qS} = 10 \frac{(\tau - \tau_c)}{\gamma (s_s - 1) d} \qquad (22)$$

The plottings of this equation with representative data are given in figure 12.

Einstein[2] departed from the previous bed load formulae and developed a new function. He assumed the probability that any one particle that would begin to move in a given unit of time could be expressed in terms of the rate of transport, the size and the relative weight of the particle, and a time factor equal to the ratio of the particle diameter to its velocity of fall. The same probability was expressed again in terms of the ratio of forces exerted by the flow, to the resistance of the particle to motion. The two forms of probability relationships were then equated to yield the generation function

$$\Phi = f \ (\Psi') \qquad (23)$$

in which

$$\Phi = \frac{q_s \ s_s}{\sqrt{g \ (s_s - 1)} \ F d^{3/2}}$$

$$\Psi' = \frac{\gamma (s_s - 1) d}{\tau}$$

$$F = \sqrt{\frac{2}{3} + \frac{36 \nu^2}{g d^3 \ (s_s - 1)}} - \sqrt{\frac{36 \nu^2}{g d^3 \ (s_s - 1)}}$$

The plotting of experimental results from various investigators for sediments of uniform grain and sand mixture according to the $\Phi - \Psi'$ function are shown in figures 13 (*a*) and 13 (*b*). For sediments of uniform grain size, the fact that results from various workers follow a curve is remarkable, since the material used in these experiments varied from 28.6 to 0.35 mm in diameter and the depth of flow varied from 0.06 to 3.6 ft. Results for non-uniform material (with representative diameter taken as that diameter for which 40 per cent of the material is finer), however, display a rather unpredictable scatter for reasons which have not yet been adequately explained.

1. Johnson, J.W., Discussion of Paper by L.Y. Chang "Laboratory Investigations of Flume Traction and Transportation" *Transactions of A.S.C.E.* vol. 104, 1939, pp. 1287-1293.

2. Einstein, H.A., "Flow on Movable Bed". Proceedings of the 2nd Hydraulics Conference, *University of Iowa Studies in Engineering*, Bulletin 27, 1943, pp. 332-341.

Einstein carried his work farther by actually making sediment load measurements in two small streams carrying sediment load of approximately 0.68 and 0.25 mm in diameter respectively. The points obtained from field measurements follow very closely the general curves derived from laboratory measurements as shown in figure 14.

To obtain a better idea of the relation between q_s and τ, Rouse plotted the experimental measurements of Φ against $1/\Psi'$ as shown in figure 15. The logarithmic plot shown in figure appears to reduce all such data to a single linear function except in the lowermost zone where the data tend to approach the same limit as those of Shields. The function according to Rouse reduced to:

$$\frac{q_s \; s_s}{\sqrt{g \; (s_s - 1)} \; Fd^{3/2}} = 40 \left(\frac{\tau}{\gamma \, (s_s - 1) \, d} \right)^3 \qquad (24)$$

$$\text{or } \Phi = 40 \left(\frac{1}{\Psi'} \right)^3 \qquad (25)$$

(*c*) *Effect of rate of transport on roughness coefficient N*

In order to use any bed load formulae, one must know the roughness of the channel for various rates of bed load transportation so that the discharge may be calculated from a known value of hydraulic radius or vice versa. It has been shown in section *sediment transportation in closed conduits* that friction ε of sediment flow in pipes increases with the rate of transportation of sediment (concentration) and a number of workers have reported the change of roughness N with the rate of sediment transportation. Einstein plotted Φ against the ratio N_s/N in which N_s is the Mannings roughness for no transportation and N the roughness at any given rate of transportation and showed that roughness is a function of Φ. The value of N_s is to be calculated by Strickler's formula:

$$N_s = 0.0132 \; d^{1/6} \qquad (26)$$

where d is the representative diameter of sand particles in millimeters which, according to Einstein, is found to be equal to the diameter of that material 65 per cent of which was smaller.[1] The curve is shown in figure 16.

Suspended load transportation

It has been pointed out before that the transportation of sediment in suspension can be considered as an advanced stage of bed load transportation, whereby particles in saltation are caught by the upward component of the turbulent velocity and are kept in suspension. Sediment transportation in suspension is, therefore, always accompanied by bed load transportation if the suspended load is derived from materials of the river bed[2] and it is very difficult to differentiate between the two in the region near the bed, where sediment particles are also in saltation. Summarizing the researches so far conducted by various authors, it can be said that attempts have been made, firstly, to clarify the problem of distribution of suspended load in the stream, and secondly, to correlate the suspended load with the bed load or river bed material. The total load (both suspended and bed load) is then to be properly correlated with hydraulic factors (slope, discharge etc.) of flow. Up to now, only the first part of the objective, namely to clarify the distribution of suspended load in a stream, has been achieved to some extent. As described before some research on sediment flow in closed conduits has also been made with a view to ascertaining the rate of sediment transport in terms of hydraulic factors but without going into the internal mechanism of sediment (suspended and bed load) transportation. The extension of such studies on open channels with erodible beds is, therefore, deemed necessary.

(*a*) *Distribution of suspended load in a vertical*

If the concentration of sediment at some height above the bed is denoted by c, expressed in weight of sediment per unit volume of fluid, and the settling velocity of particles by w, the rate at which the materials settle through a unit area at a height y will be wc. If equilibrium exists, this rate will be balanced by the rate of upward movement due to turbulent mixing and therefore

$$wc = -E \frac{dc}{dy} \qquad (27)$$

where E is the so called "exchange coefficient" which has the dimension of velocity times length. The value E is not a constant throughout the section, and can be determined for momentum transfer from the velocity distribution by the formula

$$\tau = \rho E \frac{dV}{dy} \qquad (28)$$

where τ is the shear stress, ρ the density of fluid and V the velocity of flow at a distance y from the bottom. Again, by assuming the von Karman logarithmic velocity distribution law,

1. The formula originally given by Strickler is $N_s = 0.015\, d$ where d is the mean diameter of sand particles.

2. This case also applies to clayey and colloidal material subject to sheet or small gully erosion in head water area of basin. As the muddy flow is discharged into the main stream of river with a sudden change of the hydraulic factors of flow, the clayey and colloiddal material has practically no bearing on the composition of material from the river bed. They are usually classified in the main stream of river system, as wash load.

$$\frac{V - V_{max.}}{\sqrt{\frac{\tau_0 g}{\gamma}}} = \frac{2.3}{K} \log \frac{y}{d} \tag{29}$$

E can be calculated from equation (28). The integration of Equation (27) gives the following expression

$$\frac{c}{c_a} = \left(\frac{D-y}{y} \quad \frac{a}{D-a}\right)^z \tag{30}$$

$$z = \frac{w}{K\sqrt{gDS}}$$

where c_a is the concentration at a distance "a" from the bed, and K the von Karman universal constant for turbulent flow which is about 0.4 for clear fluid. This equation was first published by Rouse in 1936.

Equation (30) gives only a relative concentration and therefore does not make possible the calculation of the total transport of the sediment. To make this possible, the value of c_a at some elevation "a" from the river must be known. In discussing the magnitude of c_a von Karman states that it probably depends on the size of sediment and on the magnitude of the tractive force acting on the river bed.

It will be noted that as z becomes small, indicating sediments with small falling velocity (size), or flow with large tractive force, the concentration distribution tends to be more uniform over the entire depth, while for high value of z, the concentration vanishes in the upper part of the stream and most of the material is carried near the bed.

The form of equation (30) was checked by Vanoni[1] in laboratory experiments and some of the results are shown in figure 17 and 18. For $d = 0.16$ mm and $z = 1.03$, experimental result checked very well with the theory. While the general form of the equation is correct for $d = 0.10$ mm and $z = 0.423$, the constant given by the theory did not agree with experiment. The discrepancy may be due to the slipping between the fluid and the sediment as the latter is accelerated by the random turbulent fluctuation of flow, as well as the difference of exchange coefficient E from the coefficient of momentum transfer of equation (28).

In studying the hydrology of the Yangtze river, Hayami[2], starting with tensor analysis of turbulent flow, arrived at the equation of concentration distribution as follows:

$$\frac{c}{c_a} = e^{-\frac{w}{\beta\sqrt{gDS}}\phi(\eta)} \tag{31}$$

$$\phi(\eta) = \int_{\eta}^{0.999} \frac{\sqrt{1+\eta^2}}{(1-\eta^2)\sqrt{\eta}} d\eta$$

$$\eta = \frac{y}{D}$$

where c_a is the concentration at river bottom (with $y = 0.9999$ D, when y is taken from water surface downward) or approximately equal to the concentration just above the "laminar boundary layer". The concentration distribution curve is similar to that given by the equation of Rouse with the exception that concentration at water surface possesses a definite value. Assuming that the concentration can be taken as nil when c/c_a is smaller than 10^{-5}, Hayami calculated the y/D values for various values of $w/\sqrt{g\,DS}$ when c/c_a equals 10^{-5}. From this he found that for $w/\sqrt{g\,DS} \geq 0.3$ the distribution of sediment is limited to the portion below $y/D = 0.86$, and for $w/\sqrt{g\,DS} \leq 0.2$ sediment is distributed more or less over the entire depth. He therefore suggested a value of $w/\sqrt{g\,DS} = 0.3$ for demarcation between suspended and bed load. For stable channels Hayami assumed that the composition of river bed material should be proportional to the sediment below 0.86D, and derived a bed material composition curve as a function of $w/\sqrt{g\,DS}$. Measurements taken along the lower Yangtze seem to be in accord with his theory, with the exception of very fine particles (see figure 19).

The theoretical bed sediment compositions for values of $w/\sqrt{g\,DS}$ and results of measurements in Yangtze are shown in figure 19.

(*b*) *Relation of suspended load to river bed composition*

It has been pointed out before that formulae developed by various authors for the distribution of sediment give only relative concentration and cannot be used to determine the total suspended load unless the concentration c_a at a certain depth is known.

Lane and Kalinske developed an expression defining the relation between sediment concentration above the bed in terms of the composition of the bed as follows:

$$\frac{w}{\sqrt{g\,DS}} = f\left(\frac{c}{c_b}\right) \tag{32}$$

1. Vanoni, V.A., *op cit.*, pp. 67-133.
2. Hayami, S., "Hydrological Studies on the Yangtze River, China," *Journal Shanghai Science Institute, New Series.* vol. I, No. 1, 1941.

where c is the concentration in p.p.m. "just above the river bottom" and c_b the percentage of particles of the total river bed material having a terminal fall velocity w.[1] This is a step in the direction of calculating the total load of a stream in terms of the hydraulic factors. Measurements taken on the Mississippi, the Missouri and the Colorado rivers in the United States, and at Tisza, Hungary,[2] shown in figure 20, seem to indicate a definite relation between these two function within the range of $w/\sqrt{g\,DS}$ from 0.02 to 1.0. The exact level at which concentrations are taken is not given. This has an important bearing in this respect that rapid change of concentration of sediments of larger size takes place near the bed.

Unified method of calculating total load

With a view to calculating the total load transported in a stream, including the bed load and suspended load, Einstein[3] developed a very interesting method to solve this complicated problem. For any given bed material and under certain prevailing tractive force of a stream, the rate of bed load transportation can be ascertained by means of using any suitable bed load formula. Einstein then correlated the concentration of the suspended load of the flow at a distance just above the "bed layer" with the rate of bed load transportation. With this known value of concentration at a distance equal to the thickness of the "bed layer" from the river bed, the distribution of suspended sediment along the entire depth can be calculated by using equation (30). The total suspended load can be obtained by integrating the product of two curves along the depth, namely, the sediment distribution and the velocity distribution curve. The total suspended load, when added to the bed load, will be the total sediment transported by the stream under prevailing conditions of flow and composition of bed material.

The basic approach to this problem rests on two principles namely (1) the restriction of bed load to the bed layer and (2) the co-relating of suspended load concentration at the surface of the bed layer to the concentration or the rate of transport of bed load. The co-relation of suspended load to bed load implies that the former is, as a result of momentum transfer or mixing, derived from the latter, and consequently from the river bed itself. Thus fine particles of any sediment load, which do not appear in the composition of the river bed material, will not be included in the computation. For practical purpose, Einstein suggested that one may even exclude the finest 10 per cent (by weight) of the river bed material since these particles do not usually represent a structural part of the bed but only loosely fill the pores between the larger particles.

The suspension of sediment particles in flow is the result of momentum transfer of small masses of fluid. The distance of travel of small masses of fluid is commonly known as mixing length, l_e. The mixing length becomes smaller as the bed or any wall is approached. The flow layer at the bed, in which mixing length is so small that suspension becomes impossible, has been found, according to Einstein, to be about 2 grain diameters thick. This is designated as the "bed layer".[4] Assuming that all the bed load material moves with an average velocity of U_B within the "bed layer" having a thickness equal to 2d, the average concentration of the fraction of bed load within certain size interval of the total bed load expressed in weight of sediment per unit volume of mixture, is:

$$\frac{i_B\ q'_B}{U_B\ 2d}$$

where q'_B is the total bed load transportation expressed in weight per unit time and unit width of channel, and i_B the fraction of bed load of certain size or size interval.

The concentration of suspended load corresponding to the size interval, also expressed in weight of suspended sediment per unit volume of mixture, is related to the average concentration of bed load by the simple relation

$$c_a = A_1\ \frac{i_B\ q'_B}{2d\ U_B} \qquad (35)$$

where A_1 is assumed to be a constant.

The velocity U_B is not known. Assuming U_B to be directly proportional to the shear velocity

$$U_* = \sqrt{\frac{\tau g}{\gamma}} = \sqrt{gRS}$$

at the bed, the above expression will be

$$c_a = A_2\ \frac{i_B\ q'_B}{2d\ U_*} = A_2\ \frac{i_B\ q'_B}{2d\ \sqrt{gRS}} \qquad (34)$$

1. Lane, E.W. and Kalinske, A.A., "The Relation of Suspended to Bed Load Materials in Rivers", *Transactions of the American Geophysical Union*, IV, 1939, p. 637.

2. Bogradi "Solid Transportation by River" La Houille *Blanche*, No. 2, 1951, p. 129. (Measurements obtained at Tisza appear to scatter on the left hand side of the curve suggested by Lane and Kalinske. Bogardi attributed this scattering to the fact that the Tisza river is not in a stable condition but shows signs of deepening.)

3. Einstein H.A.: "The Bed Load Function for Sediment Transportation in Open Channel Flows", US Department of Agriculture, Soil Conservation Service, *Technical Bulletin* No. 1026, September 1950, Washington, D.C.

4. In reality, the region in which suspension degenerates is not sharply defined. There exists rather a gradual transition to the rest of the flow. It is possible, nevertheless, to idealize the conditions by introducing a sharp division between the bed layer and the bulk of the flow as in the case of laminar sublayer.

The dimensionless constant A_2 must be determined experimentally. According to Einstein, the average value of A_2, based on special set of 26 experiments using six different sand mixtures, can be taken as $A_2 = (1/11.6)$. With this value known, the total suspended load over the entire depth for various size intervals of sediments that are present at the bed can be calculated. Einstein's approach of co-relating bed load with suspended load is indeed a very promising step toward the solution of this highly complicated problem. It would be an extremely valuable contribution if the value of A_2 can be, verified by other research workers, both on field measurements and experiments.

ABRASION AND SORTING

It is a common phenomenon that the size of river bed material gradually diminishes from head water to the sea and the same phenomenon is observed in many irrigation canals. The causes of this, as pointed out by many authors, are sorting and abrasion.

Sorting occurs with a change of the sediment carrying power of the stream. Whenever the tractive force of a stream decreases, as a result of flattening of slope, or widening of cross-section, or heading up by a structure or other obstructions, the coarser particles of sediment normally carried by the stream will cease to move and will deposit on the river bed. Sorting can take place adruptly or gradually according to the nature of the change of sediment carrying power of the stream. In the case of a torrent of steep slope discharging into a river of milder slope, an alluvial cone of gravel and boulder will be formed at the confluence of the torrent with the river. Deposition of coarser particles may be found downstream of the diversion of a stream when the flow is partly diverted reducing the sediment carrying power of the stream. Sorting can also take place gradually in natural stream or artificial canals when the discharge is gradually reduced on account of withdrawal of flow for irrigation or due to seepage and evaporation losses.

Abrasion refers to the diminishing of particle size of sediment due to the rubbing, grinding or crushing of particles against each other or against the river bed. Strictly speaking, the effect of abrasion can only be accurately accertained when the sediment carrying power of the flow is *constant* within the stretch of river under consideration. In natural streams or artificial canals, where there is no abrupt change of the condition of flow, both sorting and abrasion take place simultaneously and it is difficult to differentiate the relative importance of one over the other. Common reasoning favours the opinion that, if suspended load constitutes the major portion of the total sediment load carried by a stream, sorting is a more important factor. On the other hand if bed load is in greater proportion than suspended load, abrasion controls the decrease of sediment size as it travels downstream.

The law governing abrasion was first suggested by H. Sternberg and later verified by A. Schocklitsch from a series of experiments. Sternberg assumed that the loss of weight dp of a sediment particle is proportional to its weight p and the distance ds that it travels. The differential equation is thus:

$$dp = -c\ p\ ds \tag{35}$$

Integrating, we obtain

$$p = p_o\ e^{-cs} \tag{36}$$

where p_o is the initial weight, p the weight after the particle has travelled a distances, e the base of natural logarithm, and c a coefficient having the dimension of km^{-1} or mi^{-1} or other units if the distances is expressed in kilometres or miles, or other units. The correctness of the expression given in equation (36) has been verified by Schocklitsch from a large number of experiments.

If the diameter of a particle d is used instead of its weight p, the equation becomes

$$d^3 = d_o^3\ e^{-cs} \tag{37}$$

or

$$d = d_o\ e^{-\frac{cs}{3}} \tag{38}$$

The coefficient c, commonly known as coefficient of abrasion, is dependent on the form and the quality of the sediment (in respect to parent rock material which determines the hardness, brittleness, etc.), the velocity of sediment movement as well as the diameter of sediment composing the bed over which the particle under consideration moves. On the basis of the results of experiments Schoklitsch[1] suggested the following expression

$$c = c_1\ V^{\frac{1}{4}} \left(\frac{d + 15}{15}\right) \tag{39}$$

where V is the velocity in m/sec at which the sediment moves, d the diameter expressed in mm of the bed material over which the sediment particle moves and c_1, the specific coefficient of abrasion, is the coefficient for the case in which V equals 1 m/sec and d approaches zero, i.e., extremely fine bed material.

Schoklitsch obtained by tests the c_1 value for the 84 different kinds of sediment he had tested, a selection from which is given in table 8.[2]

1. Schoklitsch, A., *Handbuch des Wasserbaus*, 2 Aufl., Springer-Verlag, Wien, 1950, 1st Bd., p. 163.
2. *Ibid.*, pp. 163-164.

Table 8

Specific coefficient of abrasion C_1 (in km^{-1}) for different kinds of rocks moving at a velocity of 1 m/sec over a sediment bed with sediment diameter equal to zero

Serial No.	Rock	Round particles	Sharp corner particles
1.	Talc slate	0.27 — 0.23	0.30
2.	Sandstone	0.041	—
3.	Brick bat	0.032	0.041
4.	Quartz-feldspar strips with hornblende	0.0088	—
5.	Marble	0.0082 — 0.0063	0.0095
6.	Lime-sandstone	0.0079	—
7A.	Limestone marl	0.0060 — 0.0038	0.0092
7B.	Limestone	0.0050 — 0.0014	—
8.	Rose-quartz	0.0038	—
9.	Gneiss (rich in quartz)	0.0033	—
10.	Dolomite	0.0026	0.054
11.	Granite	0.0026 — 0.0013	0.025
12.	Green slate with chlorite spots	0.0022	0.0028
13.	Serizite—gneiss	0.0017	0.0018
14.	Sandstone, fine grain	0.0015	—
15.	Quartz—porphyry	0.0015	0.0032
16.	Granite—gneiss	0.0013	—
17.	Silicious slate	0.00070	—
18.	Quarzite	0.00070	—
19.	Porphyry	0.00054	0.0029
20.	Grey cast iron	0.00025	0.00032
21.	Hardened steel	0.00016 — 0.000063	—
22.	Fire brick	0.000063	0.00029

Experiments on abrasion of sand were carried out by the Indian Waterways experiment Station, Poona, from 1938 to 1946. A circular tank, divided concentrically into three compartments was filled with a layer of sand 4 inches thick and covered with water. Vertical blades fixed to a central revolving shaft were used to propel water in each compartment. A summary of the results obtained[1] is given in table 9.

Table 9

Mean diameter of sand after different periods of testing for abrasion

(original mean diameter of sand 0.2537 mm)

Period of working in hours	Mean diameter of sand in mm from different compartments with velocity of flow equal to		
	2.38 ft/sec	4.16 ft/sec	5.93 ft/sec
1.460	0.2487	0.2438	0.2408
3.770	0.2419	0.2358	0.2328
5.650	0.2345	0.2267	0.2228
6.550	0.2317	0.2240	0.2194

The results of the experiments show clearly the diminution of particle size of sand with the increase of working time and also the effect of velocity of flow on abrasion. It is, however, not possible to calculate the values of c from experimental results as only the velocity of flow and not the velocity of moving sand was measured.[2]

In addition to abrasion due to the rolling or sliding of the sediment particle there are the actions of crushing of particles against each other, the grinding of the larger particles lying on the bed at rest by those smaller particles moving over them, which also reduce the size of sediments.

As far as bed load movement in natural streams is concerned, the diminution of particle size is the result of all the factors mentioned above. The problem is further complicated because (*i*) the bed sediment is composed of materials originating from various kinds of parent rocks (*ii*) tributaries bring sediments of different size to the main stream; (*iii*) the velocity of bed sediment movement is difficult to measure; and (*iv*) the size of the material, over which bed sediment moves, is not easily known.

1. Central Board of Irrigation, India, *Annual Report (Technical) 1947*, part I, p. 248.

2. Another problem is the sampling of sand in each compartment after a certain period of working. It can be anticipated that only sand in the top layer is set in motion while that below the top layer is at rest. Sampling should therefore be limited to the layer of sand actually in motion and not the entire layer of sand in the tank.

Results of measurements of sediment size along some rivers and canals in this region and the calculated coefficient of abrasion are given in table 10. Comparing the order of magnitude of the coefficient thus calculated with those obtained by Schoklitsch, it becomes clear that abrasion can play an important role in the diminution of sediment size. The effect of sorting, however, cannot be quantitatively determined.

Table 10

Diminution of particle size of bed sediment

Yangtze River, China

Distance from river mouth (km)	Mean diameter of river bed material (mm)	Coefficient of abrasion (km^{-1})
840	0.27	0.00228 (mean)
575	0.19	
155	0.16	

Chialing River (tributary of the Yangtze River)

Distance from the confluence with the Yangtze (km)	Mean diameter of river bed material (mm)	Coefficient of abrasion (km^{-1})
710 — 550	42.5	0.00445 (mean)
550 — 310	36.0	
310 — 90	22.4	

Upper Chenab Canal, Pakistan

Distance from intake (mi)	Discharge (cu ft/sec) Summer	Discharge (cu ft/sec) Winter	Mean diameter of canal bed material (mm)	Coefficient of abrasion (km^{-1})
5	12,000	8,000	0.37	0.00421 (mean)
10	12,000	8,000	0.33	
15	12,000	8,000	0.38	
20	12,000	8,000	0.40	
25	12,000	8,000	0.35	
30	12,000	8,000	0.38	
35	12,000	8,000	0.36	
40	9,000	7,500	0.43	
45	9,000	7,500	0.36	
50	9,000	7,500	0.32	
55	9,000	7,500	0.34	
60	9,000	7,500	0.29	
65	9,000	7,500	0.32	
70	9,000	7,500	0.31	
75	9,000	7,500	0.31	
80	9,000	7,500	0.31	
85	9,000	7,500	0.31	
90	9,000	7,500	0.33	
93	6,000	6,000	0.26	

SUMMARY REMARKS

Since the transport of sediment in suspension is, strictly speaking, merely an advanced phase of bed load transport, a truly correct bed load equation should be capable of extension to include suspended load movement. Likewise, any truly correct suspended load relationship should also indicate the total transport rate including the bed load. Because of the complexity of the problem, for the time being, one is limited to the tentative selection of the principle variables determining the total bed and suspended load from the knowledge gained from various experiments and observations.

So far as sediment is concerned, as suggested by Rouse,[1] the primary characteristic is the mean terminal fall velocity w, (which takes care of the specific weight, size and form of sediment) and the secondary characteristic is the standard deviation about the mean; the size "d" plays a role only in so far as it may determine the bed roughness. With regard to the flow, the primary characteristic is the tractive force, more appropriately expressed as the shear velocity $\sqrt{\tau g/\gamma} = \sqrt{gDS}$, which determines the capacity of flow to pick up sediment and the Reynold's number $Re = (VR/\nu)$ which indicate the degree of turbulence or suspended sediment flow in pipes. The variables may be grouped in the following form

$$\frac{q_s}{Q} = \Phi \left(\frac{K\sqrt{gDS}}{\nu}, \frac{w}{\sqrt{gDS}}, \frac{VR}{\nu} \right) \qquad (40)$$

1. Rouse, Hunter, *Engineering Hydraulics*, p. 804.

Chapter III

SILTING AND SCOURING OF CANALS

THE THEORY OF STABLE CHANNELS

An unlined earthen channel composed of coherent or non-coherent material for carrying water with or without sediment is said to be stable when there is practically no scouring of its bed and banks and there is no deposit from the sediments in flow, when considered over a long period of time. The stability of a channel is essentially a problem of (1) the resistance of the material composing the channel against the erosive force of the flow and (2) the ability of the flow to carry certain quantity of sediment load of a certain size without causing significant deposition. As the quantity of flow as well as the sediment load change from time to time, any earthen channel may be subject to periodic erosion or deposition, but if the profile and slope remain more or less the same over a long period of time, say one or a few years, the channel is considered stable. The term "regime" is often used in place of "stable" in Burma, India and Pakistan.

When viewing the problem from the stand point of erosion and deposition, Thomas[1] and later Lane[2] have rightly classified unstable channels into three categories: (1) channels where the banks and/or bed are scoured without objectionable deposits being formed (2) channels in which objectionable sediment deposits occur without scour being produced, and (3) channels in which both scour and deposits are present.

An extreme case of the first category is the flow of clear water in channels of loose and non-coherent material. This is essentially a problem of tractive force of flow and resistance of the materials composing the channel. An extreme case of the second category is heavy sediment laden flow in a lined canal, where the problem of erosion is practically non-existant. Most complicated and of common occurrence in countries of Asia and the Far East is the third category where heavy sediment laden flow is carried in channels composed of easily erodible material. This is particularly true of irrigation channels in alluvial plains, where the plain itself is built up by deposits from flood flow. The well known formulae of Kennedy, Lacey and of other workers in India and Pakistan all relate to channels of the third category. Such channels will be discussed separately in a subsequent section.

1. Thomas, A.R., *Annual Report of the Central Board of Irrigation*, India, 1945.
2. Lane, E.W., *op. cit.*

Channels with comparatively clear water flow

The design of channels with clear water flow, or with turbid water containing very fine sediment which has the effect of providing some cementing material to the canal without causing any noticeable deposition, has been fully investigated by Lane and his associates.[3] The design is essentially a problem of balance between the tractive force of the running water and the resistance of material forming the canal. To be more precise, for canals of trapezoidal cross-section, the problem is to ascertain the maximum tractive force exerted on the perimeter and the least resistance of the material against erosion, which occurs usually somewhere on the side of the canal rather than at the bottom.

The mean tractive force, τ, of running water in a canal is given by the usual formula: $\tau = \gamma RS$, where γ is the specific weight of water, S is the energy gradient or in uniform flow the channel slope which of course is equal to water surface slope, and R is the hydraulic radius of the section. The distribution of tractive force along the perimeter of a canal has been worked out by Olsen and Florey,[4] using the method of membrane analogy. For trapezoidal and rectangular cross-sections, the maximum tractive force occurring on the bed and on the sides of a canal, expressed in terms of tractive force calculated from γDS (with D being the depth of flow), is reproduced in figure 21 and table 4, for various ratios of bottom with to depth and for various side slopes.

(a) *Coarse, non-cohesive material*

Regarding the resistance of material against erosion, for coarse and non-cohesive material as stated above, the limiting condition is governed by the particles on the sides rather than those

3. *Ibid.*
4. Olsen and Florey, *op. cit.*

at the bottom, as the resistance of the former is greatly reduced by the sliding force of particles on an inclined plane due to their own weight. For convenience in design, Lane expresses the effect of side slopes as a factor K, which is the ratio of the tractive force required to initiate motion of a particle on the sloping sides to that on a level bottom. The ratio K involves the angle of side slope ϕ and the angle of repose θ of the material, the functional relation of which is given in equation (41) and is also shown in figure 22.

$$K = \cos\phi\sqrt{1 - \frac{\tan^2\phi}{\tan^2\theta}} \qquad (41)$$

The critical tractive force for coarse non-cohesive material on a level bottom can be expressed by the conventional expression $\tau_c = C\gamma\,(s_s - 1)\,d$ where d is the mean diameter of particles, s_s is the specific gravity of sediment and C is a constant. Lane, after critical analysis of available data, recommended a value of 0.06 for C. With specific gravity of sediment s_s equal to 2.65, the relation between τ_c and d is shown in figure 23 in which d is the diameter of particles of which 25 per cent of the material is larger.

An example is given to illustrate the procedure in the design of a canal with coarse angular and non cohesive material, 25 per cent of which is 25 mm in diameter or over.

If W	= bottom width	= 5 m
D	= depth	= 2.5 m
R	= hydraulic radius	= 1.54 m
1:n	= side slope	= 1:2

Then for condition limited by the movement of material in the bottom of the canal

Max. tractive force on bottom
(figure 21) $= 0.89\ \gamma DS$ ton/m²

Critical tractive force τ_c for $d = 25$ mm
(figure 23) $= 2.00 \times 10^{-3}$ ton/m²

Limiting slope of canal

$$S = \frac{2.00}{10^3 \times 0.89 \times 1 \times 1.54} = .00146$$

For condition limited by the movement of material on the side of the canal

Max. tractive force on side
(figure 21) $= 0.76\ \gamma DS$ ton/m²

Safe angle of repose ϕ (figure 22) $= 36°$

K for $\phi = 36°$ and $\tan\theta = 1{:}2$
(figure 22) $= 0.64$

Allowable critical tractive force on side $=$ $0.64 \times 2.00 \times 10^{-3} = 1.28 \times 10^{-3}$ ton/m²

Limiting slope of canal $=$

$$\frac{1.28}{10^3 \times 0.76 \times 1 \times 1.54} = 0.00108$$

which is small than 0.00146.

(*b*) *Cohesive material*

When the canal is constructed in cohesive material, owing to cohesion, the particles are prevented from rolling down. Hence, the design involves the distribution of tractive force over the sides and bottom of the canal as well as the determination of the critical tractive force which the cohesive material can withstand. The value of maximum tractive force, expressed in terms of the value γDS occuring on the bottom and the sides, can be taken from Figure 21 as already explained above. No definite value of the critical tractive force can be given to cohesive clay as cohesion varies with the compactness of the clay and its composition. Values recommended by various authors vary from 0.02 lb/sq ft (0.09 kg/m²) for loose clayey loam soils to 0.6 lb/sq ft (3 kg/m²) for very compacted sandy clay.

(*c*) *Fine, non-cohesive material*

Canals in fine, non-cohesive material are intermediate between the two classes mentioned above. The presence of fine sediments or colloidal particles in the flow as well as the presence of a little binding substance in the canal material itself will undoubtedly add some cohesion to the non-cohesive material. In the design of a canal, the rolling effect of the particles due to their own weight on the sides of the canal is generally neglected. The critical tractive force that will set the particles into motion also varies with the colloidal content of the flow due to their cementing effect. The values recommended by Lane are also plotted in figure 11 in which d is the mean diameter for which 50 per cent of the weight is larger. Comparing this curve with that for coarse non-cohesive material, a discontinuity is observed in the vicinity of $d = 5$ mm. Lane suggested to remove their difficulty by taking the lower value of the two in the vicinity. However, he recommended a different criterion for particles larger than 5 mm in diameter, so that the size particles of which 25 per cent of the material is larger is taken as the effective diameter, instead of 50 per cent for the smaller size.

Channels with sediment-laden flow

The design of the second category of channels subject to sediment deposition without scouring, and the third category of channels subject to both deposition and scouring, are essentially problems dealing with sediment laden flow. In fact, the second category of stable channels can be considered as a special case of the third category in which only the material forming the channel is more resistant to erosion that the latter. If equilibrium exists on a channel of the third category, i.e., there is no scouring and no silting, it will also hold true for a channel of the second category provided the sediment laden flow is the same in both cases.

The significance of non-silting and non-scouring channels lies mainly in the design of irrigation canals, which receive their flow as well as sediment charge from rivers. Sediments carried by the main river, in particular the larger size sediments, are prevented, as far as possible, from entering the canal by proper location of the head works or by the installation of a silt excluder or/and ejectors, etc. Sediments getting into the canal usually derive their origin from the suspended load in the river, in particular during floods. As the velocity of flow in the canal is slackened, the relatively coarser particles will tend to deposit in the canal. A stable canal should be able to transport all this relatively coarser material, usually in the form of bed load, at a rate equal to the rate of the coarser sediments entering the canal, which depends on the river stage, trap efficiency of the desilting works, etc. The finer or colloidal particles of sediment which can be carried by the canal flow in suspension and ultimately into the fields, will have no bearing on the problem, with the exception of those fine sediments that may deposit on the sides of a canal if the shear is not sufficient to prevent this deposition.

It can therefore be said, as far as non-silting is concerned, that the problem is essentially one of dealing with the rate of transportation of certain *sizes* of bed load material, which depends very much on the tractive force of the flow in the canal. Even if the canal is so designed as to enable the sediment to be transported in the main canal, escapes for the sediment must also be provided at the large bifurcations, where the discharge in the branches suddenly becomes much less than in the canal upstream. In fact, the decrease of discharge in the main canal and branches as water is utilized for irrigation, including losses, is accompanied by the decrease of tractive force or sediment carrying capacity of flow, and part of the sediment is likely to be deposited, usually immeidately below the branching of the main canal.[1] Therefore, clearing of the deposit at certain intervals or a raising of the banks of the canal[2] are inevitable. If the rate of silting is not serious from a practical point of view, the canal can be considered as stable in regard to deposition. Again, the sediment rate of the river and therefore of sediments entering the canal change from season to season. Deposition may occur during flood periods where the sediment charge is high while the deposited material may be washed away during periods of comparatively clear water flow. The canal is considered stable if such variation is periodic and if equilibrium is maintained when considered over a longer period of time.

The scouring of canals depends very much on the resistance of the material composing the canal, in particular of the sides, to withstand the prevailing tractive force of the flow, which is of such magnitude as to be able to transport the required amount of sediment. In general, irrigation canals are located in river alluvium which is built up by the river itself and the composition of which is more or less similar to the sediment load of the river during floods. However, owing to the age long consolidation of the alluvium or due to artifical compaction if the canal is constructed in a filling section, the tractive force required to initiate the canal material into motion is generally larger than that required to transport the non-cohesive particles of the sediment entering the canal at a normal rate. If the sediment to be transported is much coarser than the material of the canal, the canal bottom will be covered by the coarser sediment, which again governs the condition of scouring. In such cases, however, no stability can be attained at the sides of the canal unless they are paved. An example of this is the upper Bari Doab Canal in the Punjab (India). The canal material is clay-loam and the sediment carried by the canal is shingle and gravel. The bottom of the canal is covered with shingle and the sides are paved with the same or coarser material obtained from the bed and elsewhere.

It can therefore be said that the problem of a non-scouring and non-silting canal can be simplified into the problem of designing a canal capable of transporting the sediment load of a certain size at a certain rate with a certain discharge. It is essentially a problem of bed load transportation.

(*a*) *Formulae of Kennedy, Lindley, Lacey, Bose and Inglis*

Contribution to the knowledge of stable channels came largely from India and Pakistan,[3] in particular from the Punjab. One of the earliest contributions is the classic formula presented by Kennedy which defines the relation between critical (non-scouring and non-silting) velocity V_o in ft/sec in relation to depth of flow D (in ft) as:

$$V_o = CD^m = 0.84\ D^{0.64} \qquad (42)$$

where m is an exponent.

1. By the proper selection of the slopes and sections of the canals and laterals and providing the proper sediment selective heads for the laterals, a canal system can be designed to carry through it and out onto the lands all the sediment which enters through its headworks. In other words, it must be designed to transport the sediment, just as it is designed to carry the water. This is difficult to do now as our knowledge of the laws of sediment transportation is imperfect but this probably will be remedied in the near future. In some cases it may not be desirable to construct canals this way as it may be better to run them in flatter slopes to secure "command" but deposits in irrigation canals are not necessarily "inevitable".
2. The practice adopted in India is to use the sediment cleared from the canal for raising the banks along the canal. The latter is then covered with blankets of clay.
3. Details about the historical development of regime theory are given in the following articles: (1) Inglis, C.C. and Joglekar, D.V., "Behaviour and Control of Rivers and Canals", part I., Central Water Power Irrigation and Navigation Research Station, Poona, *Research Publication No. 13*, 1949; (2) Malhotra, S.L. and Ahuja, P.R., "A Review of Progress on Theory and Design of Stable Channels in Alluvium", in *Proceedings of the Regional Technical Conference on Flood Control in Asia and the Far East* (ECAFE, Flood Control Series No. 3); and (3) Bhandari, N.N., "The Past and Future of Detritus Load Research", *International Association for Hydraulic Research Proceeding*. 1951.

Kennedy recognized that the grade of sand played an important part in this relationship and rewrote his formula in the form:

$$V = 0.84\ CD^{0.64} \qquad (43)$$

where C is a factor affording a correction for the size of sand or silt.

In 1919 Lindley proposed a set of formulae including the surface width of canal B (in ft) but not attempting to include the effect of sediment, as:

$$V = 0.95\ D^{0.57} \qquad (44)$$

$$V = 0.57\ B^{0.355} \qquad (45)$$

$$B = 3.80\ D^{1.61} \qquad (46)$$

Most important is perhaps the contribution of Lacey who, having defined the relation between many variables, introduced a silt factor f and the following set of formulae:

$$P = 2.668\ Q^{\frac{1}{2}} \qquad (47)$$

$$R = 0.475\ Q^{1/3}/f^{1/3} \qquad (48)$$

$$A = 1.26\ Q^{5/6}/f^{1/3} \qquad (49)$$

$$V = 0.794\ Q^{1/6}/f^{1/3} \qquad (50)$$

$$S = 0.000\ 547\ f^{5/3}/Q^{1/6} \qquad (51)$$

$$f = 8\sqrt{d} \qquad (52)$$

where d is the diameter of the sediment particles expressed in millimeter, P the wetted perimeter in ft, R the hydraulic radius in ft, A the cross-sectional area in sq ft, V the velocity in ft/sec, Q the discharge in cu ft/sec, S the slope and f the silt factor.

Bose and Malhotra,[1] after several years of painstaking collection and statistical analysis of data, derived the following formulae using f.p.s. units, except for d;

$$P = 2.68\ Q^{\frac{1}{2}} \qquad (53)$$

$$S = 0.00209\ d^{0.86}/Q^{0.21} \qquad (54)$$

$$R/P = S^{\frac{1}{4}}/6.25\ d \qquad (55)$$

where d is the weighted mean diameter of the sediment in mm.

Both the silt factor f of Lacey and the weighted mean diameter d of Bose, define the size of the sediment transported in a stable channel, but not the sediment "charge" or rate at which the sediment is transported. It can be anticipated, therefore, that these formulae are applicable to canals that carry sediment at approximately the same concentration as the canals in India and Pakistan from which the formulae were derived. In India and Pakistan, great care is given to the operation of head regulator in accordance with prevailing conditions to avoid excessive rate of sediment going into the canal. Instances are on record in India when considerable stretches of a canal got silted up within a night when the head regulator was not properly operated during a flood carrying heavy sediment. This shows the importance of *rate* of transportation to the stability of canals.

To take care of the sediment "charge", Inglis[1] introduced a set of dimensional homogeneous equations. The constants involved have not yet been determined. Inglis explained further that the sediment "charge" has small effect on the area of a channel, relatively great effect on slope and shape and considerable effect on the width of channel. These formulae, together with the corresponding ones of Lacey, are listed below:

Inglis' formulae

$$B = C_1 \frac{Q^{\frac{1}{2}}}{g^{1/3}\ \nu^{1/12}} \left(\frac{cw}{d}\right) \qquad (56)$$

$$A = C_2 \frac{\nu^{1/36}\ Q^{5/6}}{g^{7/18}\ (dcw)^{1/12}} \qquad (57)$$

$$V = C_3 \frac{g^{7/18}}{\nu^{1/36}}\ Q^{1/6}\ (dcw)^{1/12} \qquad (58)$$

$$D = C_4 \frac{\nu^{1/9}}{g^{1/18}} \frac{Q^{1/3}\ d^{1/6}}{(cw)^{1/3}} \qquad (59)$$

$$S = C_4 \frac{(dcw)^{5/12}}{\nu^{5/36}\ g^{1/8}\ Q^{1/6}} \qquad (60)$$

$$\frac{B}{D} = C_5 \frac{Q^{1/6}\ (cw)^{7/12}}{g^{5/18}\ \nu^{7/36}\ d^{5/12}} \qquad (61)$$

Corresponding Lacey's formulae

$$P = 2.67\ Q^{\frac{1}{2}} \qquad (47)$$

$$A = 1.26\ \frac{Q^{5/6}}{f^{1/3}} \qquad (49)$$

$$V = 0.7937\ Q^{1/6} f^{1/3} \qquad (50)$$

$$R = 0.4725 \left(\frac{Q}{f}\right)^{1/3} \qquad (48)$$

$$S = 0.000547\ \frac{f^{5/3}}{Q^{1/6}} \qquad (51)$$

$$\frac{P}{R} = 5.65\ Q^{1/6}\ f^{1/3}$$

1. Bose, N. K. and Malhotra, J.K., "An Investigation of the Inter-relation of Silt Indices and Discharge Elements for Some Regime Channels in the Punjab", Punjab Irrigation Research Institute, Lahore, *Research Publications*, vol. II, No. 23, 1939.

2. Inglis, C.C., "The Effect if Variations in Charge and Grade on the Slopes and Shapes of Channels", *Proceedings International Association for Hydraulic Research*. Third Meeting, Grenoble, France, 1949 and Inglis, C.C. and Joglekar, D.V., op. cit., part I, pp. 186-187

In the foregoing equations $c = \frac{Q_s (s_s - 1)}{Q}$ where Q_s is the rate of sediment transport expressed in volume per sec, s_s the specific gravity of sediment, Q the water discharge, w the terminal fall velocity of sediment in water, ν the kinematic viscosity and g the gravitational constant.

The set of formulae presented by Inglis is the only one developed in India and Pakistan which has taken into consideration the sediment charge. Unfortunately the constants involved have not yet been determined and the validity of the formulae have not yet been verified by taking actual sediment discharge measurements in canals which are considered stable. It is regretful to note that the subject of stable channels which was developed almost exclusively in India and Pakistan has been completely neglected in recent years. Design of new canals are based on the formulae of Kennedy, Lacey or Bose and disputes among engineers in India on the choice of the various formulae are not uncommon. Considerable practical experience is always required before an engineer is able to choose suitable coefficients for the particular formulae which he prefers to use. As for engineers who are not familiar with the conditions on the basis of which the formulae are applied, they are completely at a loss to make any proper use of them.

(*b*) *The tractive force approach*

It has been repeatedly pointed out that the sediment charge as well as the size of sediment are both important for the regime of canals, and of the sediment load transported, the relatively coarser particles moving as bed load are the governing factor. Instead of any formulae available for practical use that can take care of the sediment charge, reliance may be placed on the bed-load formulae developed by various authors. As explained earlier, the various bed load formulae are also empirical in nature and are valid only within the range covered by experiments or field measurements. The bed load equation of Einstein may be chosen as it includes measurements both in natural streams and laboratory flumes covering a wide range of tractive force and rate of sediment transportation as well as sediment size.

According to Rouse, the bed load function that fits very well the upper ranges of the plotting of Einstein, is

$$\frac{q_s\ S_s}{\sqrt{g(s_s - 1)}\ \varsigma\ d^{1.5}} = 40 \left(\frac{\tau}{\gamma\ (s_s - 1)\ d}\right)^3 \quad (62)$$

where q_s is the rate of sediment transportation in volume of sediment per unit width of channel or flume per second, s_s and d the specific gravity and diameter of sediment, τ the tractive force of flow, γ the specific weight of water and ς a dimensionless factor which is given by

$$\varsigma = \sqrt{\frac{2}{3} + \frac{36\nu^2}{gd^3\ (s_s - 1)}} - \sqrt{\frac{36\nu^2}{gd^3\ (s_s - 1)}} \quad (63)$$

where ν is the kinematic viscosity of water and g the gravitational constant. For $s_s = 2.65$ and $\nu = 0.01$ cm²/sec, the values of ς for $d = 1$ to 10 mm are found to be about 0.81 which can be used for all practical purposes.

From equation (62) we get

$$q_s = \frac{40\ \varsigma \sqrt{g}}{\gamma^3\ s_s\ (s_s - 1)^{2.5}}\ \frac{\tau^3}{d^{1.5}} \quad (64)$$

$$= C_s \frac{\tau^3}{d^{1.5}} \quad (65)$$

Substituting $s_s = 2.65$, $\varsigma = 0.81$, $\gamma = 1 t/m^3$ and $g = 9.81\ m/sec^2$, the value of C_s comes out to be 11.0 and hence

$$q_s = 11.0 \frac{\tau^3}{d^{1.5}} \quad (66)$$

In the above expression q_s is in m^3/sec per metre width, τ is in t/m^2 and d is also in metres. As $\tau = \gamma RS \doteq \gamma DS$ for wide shallow streams, where D, the depth, is approximately equal to the hydraulic mean radius R, the above equation can be expressed as follows:

$$q_s = 11.0 \frac{D^3 S^3}{d^{1.5}} \quad (67)$$

It is conventional that the representative diameter of sediment is expressed in mm and the numerical value of C_s becomes 349,000 and the equation takes the form

$$q_s = 349{,}000\ \frac{D^3 S^3}{d^{1.5}} \quad (68)$$

In the f.p.s. system of units, q_s is measured in cu ft/sec per ft, D in feet and d in mm and the expression becomes

$$q_s = 106{,}000\ \frac{D^3 S^3}{d^{1.5}} \quad (69)$$

The discharge in a canal can be expressed by the well known Manning's equation.

$$q = \frac{1}{N} D^{1.67}\ S^{0.5}\ \text{(c.g.s. units)} \quad (70)$$

and

$$q = \frac{1.486}{N}\ D^{1.67}\ S^{0.5}\ \text{(f.p.s. units)} \quad (71)$$

where q is the the discharge per unit width in m^3/sec/m (or ft^3/sec/ft), D the mean depth in m (or ft), S the slope and N the Manning's coefficient of roughness. Connecting the roughness N to the mean diameter of sediment by STRICKLER'S equation, i.e. $N = 0.015\ d^{1/6}$, where d is expressed in mm, we obtain

$$q = \frac{66.7}{d^{1/6}} D^{1.67} S^{0.5} \text{ (c.g.s. units)} \qquad (72)$$

and

$$q = \frac{99}{d^{1/6}} D^{1.67} S^{0.5} \text{ (f.p.s. units)} \qquad (73)$$

Combining Equation (68) and (72), or (69) and (73) we obtain

$$\frac{q_s}{q} = 5{,}200 \frac{D^{1.33} S^{2.5}}{d^{1.33}} \text{ (c.g.s. units)} \qquad (74)$$

and

$$\frac{q_s}{q} = 1{,}070 \frac{D^{1.33} S^{2.5}}{d^{1.33}} \text{ (f.p.s. units)} \qquad (75)$$

which express the rate of sediment transported in terms of depth, slope of the canal and mean diameter of sediment. For practical applications of the formula in the design of canals, the surface width B is first arbitrarily chosen. With known value of $q = Q/B$, $q_s = Q_s/B$ and d, the two unknowns D and S can be determined from the two sets of equations (68 and 72, and 69 and 73). Different sets of D and S will fit different values of B from which the most appropriate one, governed by criteria such as topographic limitation of slope, minimum excavation of cross section, width of right of way, etc., will be chosen. It should be pointed out here that this approach does not enable one to decide on the shape of the canal.

It might be of interest to compare Equations (74) and (75) with those of Lacey and Bose. Assuming that the Lacey and Bose formulae contain a factor (q_s) in their coefficients, equations (74) and (75) can be written in the form:

$$S = \text{const.} \times \frac{q_s^{2.5}}{D^{8/15}} \times \frac{d^{8/15}}{q^{2/5}} \qquad (76)$$

as compared with the corresponding slope formula of Bose which is

$$S = \text{const.} \times \frac{d^{0.86}}{Q^{0.21}} \qquad (54)$$

and Lacey's which is

$$S = \text{const.} \times \frac{d^{5/6}}{Q^{1/6}} \qquad (51) \text{ and } (52)$$

Much work still remains to be done on this complicated problem. Most important of all appears to be the measurement of sediment charge in canals, which are stable, to be supplemented by systematic experiments and investigations in laboratory flume. A more appropriate set of formulae could then be worked out.

DESILTING WORKS

The necessity of desilting works at canal intakes (headworks) has now been generally realized and such works have been constructed in many countries.

Owing to physical conditions of the land, canals have to be designed with slopes flatter than those of the rivers from which they take off so as to command the areas to be irrigated, and the volume of flow in a canal is usually much less than that in the river from which it takes off, particularly during the rainy season and in some cases, during the early hot weather also, due to snowmelt. This sudden reduction in the volume and slope of flow in the canals is the main cause which leads to silting, although sometimes bad regulation, such as the sudden lowering of the retention or pond level at the intake, may also cause large quantities of bed load to move from the river into the canal.

The physical condition of a valley often makes it impossible to design channels that will carry all the silt contained in the only water available for it. The velocity that is practicable for an irrigation canal in alluvium will generally be able to carry in suspension most of the finer sediment. Therefore, preventing coarser sediment from entering canal systems is a prime factor in the success of irrigation enterprises in that it eliminates the high annual expenditures for sediment disposal and control in the canal system and upon the land, provides a more free passage of water through canals and renders structures serviceable and operative.

Rivers in the ECAFE region and elsewhere carry very large quantities of sediment loads during the monsoon period. In some cases the Himalayan streams transport specially heavy boulders 2 to 3 ft in diameter, gravels, coarse sands, etc. During a flood, several million cubic feet of this material may be transported in a day.

Several measures are adopted to reduce the sediment load entering canals. It is usual almost everywhere to close the canal when the river discharge exceeds a specified figure or when the sediment content exceeds a certain percentage by weight. However, even below these specified limits, huge quantities of sediment of coarse material are likely to find their way into the canal system. Suitable measures for preventing entry of excessive sediment into the canal system have therefore to be adopted to maintain the channels in working order.

A lot of work has been done in this connexion in the Punjab (India and Pakistan), where silting troubles started early in the history of canal irrigation, and various methods have been developed to deal with the problem.[1]

These methods can be divided into different categories according to the site of application:

1. River approach
2. Design of headworks and training works
3. River regulation
4. Sediment excluders and extractors
5. Setting basins or silting tanks

River approach

The most efficient natural method of sediment exclusion has been found to be the taking of water from outside a curve, that is from the concave side of a river bend. The most suitable position for a canal offtake is near the downstream end of a concave bend where flow is deflected towards the opposite bank and the bed load is diverted towards the convex bank. This has been done for the Sakri canal in Bihar, where the right bank of the Sakri river was advanced downstream to the canal offtake forming a concave bend and was pitched with stone to maintain it in that position.

It is advisable to select a site with a suitable curvature and to maintain the river along that curvature. Training works have ben constructed on many rivers in the Punjab in order to secure the proper curvature of the approach channel. One of the most efficient are the training works above the offtake of the Sirhind Canal on the Sutlej at Rupar, where a series of armoured spurs have been built along the left bank to obtain the required curvature. An approach channel on a suitable curvature has also been constructed at Sukkur, Sindh, for excluding sediment from the right bank canal.

The slope in the river channel approaching the intake is also of considerable importance. As soon as the slope increases, the sediment movement in the river channel considerably increases. A rising river increases sediment movement and intensity, firstly by scouring down the bed to increase the cross section for the increased discharge and also by increasing the river slope, particularly in a flood. It was found in the Chenab river at Khanki, the headworks of the lower Chenab canal in Punjab, that a river slope in excess of about 1 in 4,000 brought in heavy sediment load into the canal. An attempt was therefore made to keep the slope between 1 in 4,000 and 1 in 5,000. This was attained by closing the canal and flushing the approach channel, if necessary, or by simply raising the water level in the river at the canal offtake.

1. Uppal, H.L., "Sediment Excluders and Extractors", International Association for Hydraulic Research, 4th Meeting, Bombay, 1951, Question 2. R.7.

Another method adopted at some of the headworks, notably at Khanki and Rasul, was to divide the river approach channel into two channels, thereby considerably reducing the discharge approaching the canal offtake and thus reducing the sediment movement.

Design of headworks and training works

The following parts of the design effect sediment conditions entering a canal:

(*a*) Type of weir or barrage
(*b*) Alignment of the regulator with reference to the river flow direction
(*c*) Sill level of the regulator
(*d*) Shape of guide banks
(*e*) Length of divide wall, separating the undersluices from the main weir or barrage
(*f*) Position of undersluices.

(*a*) *Type of weir*

It is found that narrow and deep weirs induce more favourable conditions for sediment control, while weirs which are wide and therefore shallow, induce the formation of islands and their development and erosion, frequently cause unfavourable conditions, which can be obviated to a great extent in the former.

(*b*) *Alignment of the regulator*

The canal regulator should be so aligned that a suitable curvature of flow is induced in front of the regulator. The upstream end of the regulator slightly projecting into the river gives good results as it pushes the bottom current away from the regulator. The regulator should not be set back and should preferably be at right angles to the barrage.

(*c*) *Sill level of the regulator*

It is usual to keep the sill level of the canal regulator at a sufficient height above the floor or the crest of the undersluices of the weir. In fact at many of the offtakes the sill level had to be raised subsequent to the original construction of the structure, with a view to reduce sediment entry into the canal. However, experience has shown that high sill level alone cannot prevent sediment entry into the canal. This raising does, however, give a sufficient depth of water in front so that the silting in what is known as "the pocket" can be watched and steps can be taken to improve matters before this sediment starts flowing into the canal.

(*d*) *Shape of guide banks*

The shape of the guide banks upstream of a weir or barrage helps to some extent to secure a suitable approach to the canal. The shape in each case is governed by the existing river approach. Several shapes of guide banks have been investigated in the Punjab. These are parallel, converging, and bottle-neck guide banks. The bottle-neck guide bank system adopted at Suleimanki on the Sutlej has not proved a success as large islands have formed upstream of the canal offtake. A slightly diverging type of guide bank, adopted at Kalabagh on the Indus as a result of model tests, has proved superior as the main river flow with this type is situated some distance away from the canal offtake and therefore the main sediment front has moved away. At most of the weirs the influence of guide banks is masked by the large islands that have grown in the river upstream.

(*e*) *Length and shape of divide wall*

On almost all of the Punjab headworks, divide walls are constructed to separate the undersluices from the main weir structure. The length of this divide wall and its distance from the canal offtake are very important. Experiments conducted at the Malikpur Hydraulic Station, which have not yet been confirmed in prototype, have shown that:

(*i*) The shorter the length of divide wall, the smaller was the quantity of gravel and sand entering the canal;

(*ii*) The greater the distance of the divide wall from the regulator, the smaller was the quantity of gravel and sand entering the canal.

It was also concluded that if the whole of a weir or barrage were at the same level, a divide wall may not be necessary. Where the undersluices floor is at a lower level, a divide wall becomes necessary to separate them from the higher weir level. In such case, it was found that a divide wall of length equal to two-thirds of the length of the canal regulator is most suitable for sediment control as this length helps to form a suitable curvature of flow.

Tests conducted at the Denver Hydraulic Laboratory of the Bureau of Reclamation, United States,[1] also clearly showed the advantage of divide walls in diverting coarse sediment away from the canal headworks. These tests also showed that a curved divide wall was even more efficient than straight ones. As a result of model tests two curved walls of steel sheet piling were constructed upstream of the Superior-Courtland diversion dam on Republican river, 50 ft and 100 ft in length respectively. By the installation of these minor structures, water carrying about two-thirds of the sediment is twisted away from the headworks and is carried away into the river, while relatively sediment-free portion of the stream flow is channelled into the canal. Recent studies giving more and up to date information on the subject have been carried out by Martin and Carls[2]. Curved diversion walls, therefore, appear to be one of the very promising solutions for minimizing sediment entry into irrigation canals.

1. Carlson, E.J., *New Twist in Sediment*, *Reclamation Era*, published by the Bureau of Reclamation, USA, June 1951.

(*f*) *Position of undersluices*

Sluices have been considered necessary next to the canal offtake, to maintain a well defined channel in front of it to feed the canal.

The construction of additional sluices in the middle of the river was first attempted at Khanki on the Chenab with a view to inducing the necessary curvature of flow for silt exclusion from the canal. While no definite conclusion could be drawn from the working of these sluices, it seems that they are a bit too far away from the canal offtake to be of much practical use. On a barrage, any part of it could be used for creating the curvature of flow provided it is safe from other considerations. This problem needs further investigation before the method could be adopted for general use.

River regulation

The measure, which has been found successful and is most commonly adopted for river regulation in the Punjab and elsewhere in India, is what has come to be known as "the still pond system". The main feature of this system of regulation is that the water level in the river at the canal offtake is kept at a steady level and no water is escaped through the sluices, as long as the canal is open.

Normally, the canal is closed as soon as it becomes necessary to escape the water through the sluices or the water level is required to be lowered. However, there is not much objection in raising the water level with the canal open. This is because lowering the water level results in increasing the river slope and consequently increases sediment movement in the river and sediment entry into the canal, while the raising of the water level decreases the river slope as also the sediment movement. It is usual to escape the surplus discharge through a part of the weir as far away from the canal offtake as possible.

On rivers, where it is not practicable to adopt the still pond system of regulation, such as on the river Ravi at Madhopur where the Upper Bari Doab Canal takes off and on the Yamuna at Tajewala where the western Yamuna and Eastern Yamuna Canals take off, and the sluices have necesasrily to be used for purposes of regulation, regulation of supplies to reduce

2. Martin, Harold M. and Carlson, E.J. "Model Studies of Sediment Control Structures on Diversion Dams", *Proceedings Minnesota International Hydraulic Convention*, September 1953.

sediment entry into the canal becomes an important problem. The most suitable method in such cases, from the point of view of sediment control in the canal, is to open first those sluices which are far removed from the offtake and increase the openings towards the canal only when the first ones are fully opened.

Another common measure adopted almost all over the world is the closing of the canal when the river stage is such that the water is heavily laden with sediment. The canal may also be closed when the river bed level in front of the canal offtake is so high that the sediment is likely to be carried into the canal. The exact time of closing and the period for which it should last can be judged by experience at each river and actual observations of suspended silt in the river and in the canal. Such observations should be made at least once a day for use in river regulation.

The canal has also to be closed for flushing the approach channel so as to wash out all the silt that accumulates from time to time. This is generally done just before the river starts rising in the monsoon season so as to wash the sediment and deepen the channel to carry the extra water without excessive sediment movement when the river rises. These closures should be of sufficient duration to obtain the required conditions in the approach channel. Similar closures also become necessary during the monsoon when there are frequent fluctuations in the river discharge due to rains and the approach channel shows signs of silting. Such closures usually last from 24 hours to about three days.

Sediment excluders and extractors

Deposits in canal usually settle near their upper reaches, and a common method of removing such deposits is to scour them out by allowing the water to escape from the canal through a sluice located some distance downstream of the headworks. This is sometimes done by continuously using part of the canal flow and in other cases by occasionally flushing out the deposited section with the entire canal flow.

Special arrangements called sediment excluders and extractors are structures for reducing the sediment entering a channel. Excluders which are located *upstream* of the canal head, prevent the sediment from entering the canal; while extractors, ejectors or eliminators are usually located downstream from the head regulator on the canal and extract or eject the sediment after it has entered the canal.

A great variety of devices has been used for excluders and extractors. This subject has been studied most extensively in India and Pakistan, but some work has been done in other countries also. Practically all of the devices used involve one or more of the following types of action: (*a*) the slot; (*b*) the step; (*c*) deflecting vanes; (*d*) drawing-off of slower-moving currents; (*e*) separation of top and bottom water; (*f*) the skimming platform; (*g*) grillage; and (*h*) curved currents.

(*a*) *The slot*

The slot, being an extractor, is a depressed trough or channel across the bottom of a canal into which the sediment is supposed to drop as it passes over it when flowing down the canal. This has been tried experimentally in India and Pakistan both in the laboratory and in the field. Field results showed that one slot is not capable of drawing all the sediment and there was little difference in the quality of sediment deposited in two slots placed about 20 feet apart in the bed of a small channel. Laboratory experiments were more successful and showed that a slot can be quite effective and that the water required to work it in the model was about 5 to 10 per cent of the canal discharge. With a larger model, it was found that the whole length of the slot across the channel was not effective in as much as the effect of the sluice reached only one-third of the length of the slot. Further investigations are necessary to determine the efficiency of slots in large canals.

(*b*) *The steps*

The step is somewhat related to both the slot and the settling basin, at the downstream end of which the bottom rises vertically to the normal canal bed level. A gate is placed in the canal bank just upstream of the step through which the sediment deposit in the basin can be flushed out. This has the same defects as mentioned above in the case of a slot.

(*c*) *Deflecting vanes*

Deflecting vanes are excluders placed in channels to deflect the material rolling along the bed or carried in the water near the bed, away from an opening through which water is withdrawn. This method has been extensively used in India and Pakistan and found to be very successful.[1]

(*d*) *Drawing off of slower moving currents*

Drawing off of the slower-moving currents is another means of removing the sediment from irrigation water. The slower-moving currents are usually at the bottom of the channel where most of the heavier sediment is carried, and these currents are more easily deflected than the faster currents near the surface. A sluice in the side of the channel will therefore draw more water from the slow moving bottom currents than the fast moving top ones, and the water

1. King, H.W., "Silt Exclusion from Distributaries", *Proceeding of the Punjab Engineering Congress*, vol. XXVII, 1933.

drawn out will contain much more than the average concentration of sediment carried by the canal.

(*e*) *Separation of top and bottom layers of water*

Since the concentration of sediment of larger particle size is much greater near the bottom of a canal than near the surface most of the coarse sediment can usually be taken out from the bottom by placing a horizontal disphragm across the canal some distance above the bottom and wasting the water which passes beneath the diaphragm.

(*f*) *Skimming platforms*

Skimming platforms have the same action in a small part of the channel where they are constructed as the separators. They are supposed to skim off the top water which has a smaller load of sediment and allow it to pass into the canal, while the remaining water with its heavier load of sediment flows down the stream. However, they separate out only those particles which are too heavy to be lifted up over the platform by the currents which rise from the bottom and flow over the platform.

(*g*) *Grillage*

In streams which carry very coarse material such as cobbles or boulders, the sediment is sometimes separated out by drawing the water from the bottom of the stream through a grillage of bars which separates out the cobbles and boulders and allows them to roll on down the stream.

(*h*) *Curved currents*

The action of curved currents is the same as that involved in placing the canal intake at the outside of a bend, as discussed above.

Settling basins or silting tanks

Settling basins are enlargements of the channel in which the velocity and the turbulence are reduced, and the sediment settles to the bottom from whence it can be flushed or pumped out. This type of arrangement will remove more finer sediment than any of the other types. Theoretically, as much sediment as desired can be removed from the water in a settling basin, but it is rarely practicable for irrigation purposes to remove a high percentage of particles smaller than sand (0.06 mm in diameter) from the water. None of the other devices discussed above will separate material much finer than sand.

In using the various devices for sediment exclusion and extraction, there are two very important considerations which must be kept in view. The first is that all the devices require some water for their operation, although some of them require more water than the others. The type of device applicable is therefore somewhat limited by the amount of surplus water available. Secondly the withdrawal of extra water from the river to meet the requirements of excluders and extractors increase the velocity of approach in front of the canal offtake as also in the approach channel and must necessarily increase the quantity of sediment that enters the canal. In some cases the volume of water required to operate the device may be so large that the device may not be able to dispose off even the extra quantity of sediment that is drawn from the river as a result of the extra discharge and the amount of sediment entering the canal may be even more than it would be if the device were not used. It is almost impossible to verify this phenomena by observations at site because the sediment content in a river never remains constant and the discharge also varies particularly when surplus water is available for operating these devices. It should, however, be possible to verify the results by means of model tests where both the discharge and the sediment content can be controlled.

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Chapter IV

SILTING OF RESERVOIRS

In view of the seasonal variation of rainfall in many countries of the world, particularly in most of the countries of the ECAFE region, it is essential to store water during the flood season in reservoirs, large and small, and to utilize this water during the dry season. The rainfall is generally concentrated during the few monsoon months while crops require water for irrigation regularly throughout their growing season. Similarly water is required for generation of power and for navigation throughout the year. It is only by storing water that the vagaries of nature can be met.

One of the most important problems in the development and maintenance of storage reservoirs is the loss of storage capacity due to silting. Silting results from the deposition of stream-born sediments as the transporting power of the stream is suddenly reduced when flowing into the still water of a reservoir.

All streams transport some sediment and consequently reservoirs already constructed in Egypt, India, Japan, China (Taiwan), the United States and elsewhere show a tendency to silt in varying degrees, depending upon a number of factors. Records show that the rate of silting of some of these reservoirs is alarming. For example, the Yasuoka reservoir on Tenryu river in Japan, having an original capacity of 50 million cubic metres, lost as much as 85 per cent of its capacity in 13 years. Many reservoirs in India lose about 1 per cent of their capacity annually, while some of them lose over 2 per cent each year.

Ultimately, every reservoir is bound to silt up, but the evil day can certainly be put off for sometime by taking appropriate measures before it is too late. Because of the large stakes involved, every single year that can be added to the useful life of a reservoir is abundantly worth the cost and trouble involved in its achievement.

RESERVOIR SEDIMENTATION

Sediment inflow

In order to estimate the effects of sedimentation, each reservoir must be carefully studied in the light of particular data applicable to it. First in the order of importance for analysis is the anticipated rate of sedimentation. The best basis for such an estimate would be long term records of suspended load and bed-load measurements made at, or near, the reservoir site. If such records are not available, a sampling station should be established at or near the proposed or likely reservoir sites, and the results coordinated with those of other long-term records if available elsewhere on the same or similar streams. Sediment record data, even for a short period, is certainly better than none at all. Frequent absence of sampling stations in countries of the ECAFE region and outside shows the urgent need for the immediate establishment of a coordinated network of stream-gauging and sediment sampling stations on all major streams in every country. With known daily sediment charge data, the sediment runoff W_d in tons per day may be estimated by the formula:

$$W_d = 86{,}400 \times \frac{c}{1{,}000} \times Q \qquad (77)$$

where Q is the daily mean discharge in cms and c the sediment charge expressed by weight of dry sediment in grammes per litre. When Q is expressed in cfs as Q', then

$$W_d = 86{,}400 \times \frac{c}{1{,}000} \times \frac{Q'}{35.3} \qquad (78)$$

It will, of course, be necessary to determine the specific weight of the sediment and also how much of the sediment load will pass through the reservoir without being deposited or how much is deposited, i.e. the trap efficiency of the reservoir.

A simpler method of estimating sediment discharge consists of developing a correlation between sediment load and water discharge, called a sedimentating curve. The sedimentating curve is then applied to a long-time flow-duration curve and the resulting computation represents a long-time average sediment yield. This method[1] has been considered a useful and valuable tool.

The correlation between discharge and sediment load cannot be used indiscriminately for

1. According to the US Bureau of Reclamation, the sediment-rating curve—flow-duration curve method of estimating sediment yield appears to be sufficiently accurate for practical application. When applied to the flow-duration curve the correlation between sediment yield and discharge based upon the 19 years of record at Bluff, Utah, checks with the 19-year measured quantity within 4 per cent.

computing the mean sediment load on the basis of one year of sediment record since there is no way of determining whether the year in question will follow the general trend or will have abnormal tendencies. However, since the greatest deviations occur in the dry years, estimates of mean sediment loads based on years of greater than 90 per cent of mean discharge should produce results with a maximum error of 20 per cent.

The similarity of the general shape and slope of the sediment-rating curves further establishes the validity of the use of these curves as an index of sediment load, as the widest divergency of the curves occurs during periods of low flow.

The unit weight to be used in the sediment load computations should be carefully determined. A unit weight greater than the actual results in an under-estimation of the sediment load; whereas, a unit weight smaller than the actual will result in an over-estimation. This becomes increasingly important when the computed sediment load is to be used in determining the space allocation necessary for sediment in a reservoir.

The use of daily records collected for several years of normal flow tend to bunch near the median point of flow, and at this position the curve is well defined. Additional sampling records are found desirable for the high points of flow because generally a few years of record produce very few points in this range. The same holds true for the low flows, however, generally the low flows carry very little sediment and have very little effect on the results, while the high flows carry tremendous amounts of sediment and can materially affect the results.

Location of sediment deposits

Another important problem in connection with the sedimentation of reservoirs is the location of the sediment deposits within the reservoir. In studying the sedimentation of a reservoir during its planning, an estimate of the time required to fill the reservoir with sediment is frequently made by dividing the volume of water which the reservoir will hold by the volume of sediment per year which all the streams flowing into it will carry. Similarly, the space provided for the sediment in a reservoir, called "dead storage" is also based on similar calculations. However, these are very crude approximations, because some of the sediment entering the reservoir may pass through it without deposition, while some may deposit upstream of the reservoir in the stream channel and a lot may deposit in the portion of the reservoir above the space reserved for dead storage. In order to make reliable estimates of the rate of filling of a reservoir, it is necessary to ascertain the location of the sediment deposits within the reservoir itself.

In many respects the deposits in a reservoir resemble those in a delta made by a stream discharging in a lake or sea. The three parts of delta deposits, namely fore-set beds, bottom-set beds and top-set beds are also applicable to reservoir deposits. There is also a fourth kind of deposit caused by density currents. Figure 24 represents a longitudinal section through a reservoir formed by a dam in a stream, and shows the relative position of these four kinds of deposits. A fifth kind of deposit occurs in some reservoirs due to wave action along its banks.

A delta forms at the mouth of each stream entering a reservoir, but only deposits at the lowest stage of the reservoir have a chance of remaining in place. All deposits at higher stages are gradually reworked by the streams as the reservoir falls and are therefore intermittently carried forward further and further into the reservoir, excepting the coarse material which seldom moves much into the reservoir.

Besides the fluctuations in the water level, another factor which influences sediment deposits in a reservoir is its shape. Figures 25 and 26 show the position of the bottom of the reservoir at a number of different times for Soyama and Koyadaira reservoirs in Japan.[1] As the storage space in the reservoir becomes smaller and smaller, the proportion of the fine material which escapes from the reservoir with the outflowing water increases and therefore the rate of sedimentation decreases. However, unless the reservoir is emptied periodically, very little material of larger size will pass out of the reservoir until it is practically filled up with sediment.

The great complexity of the various problems in the process of sedimentation of reservoirs and the inflowing streams; the uncertainty regarding the space occupied by sediment; lack of knowledge regarding the contribution of sediment by water and wind from the tributaries; the variability of sediment inflow from year to year and from season to season and even from day to day within the same season; the inability to evaluate accurately between suspended sediment and bed-load; all militate against drawing any definite conclusions regarding the useful life of any reservoir subject to sedimentation. It is, therefore, essential to watch the actual rate of sediment deposition after a reservoir has come into operation and this can be done only by actual measurements and survey from time to time.

Specific weight of deposited sediments

The rate of compaction of silt in a reservoir considerably affects its useful life and its future operation. Thus the useful life of a reservoir will be much more if the specific weight of the sediment is 1.2 t/m^3 (75 lb/cu ft) than with 0.95 t/m^3 (60 lb/cu ft).

1. Nippon Hassoden Kabushiki Kaisha, 1950. *Sedimentation of Principal Reservoirs in Japan.*

Sediment deposited in a reservoir varies greatly in weight and volume, depending on its source, its fineness, the depth of deposition and the degree of submersion or exposure. No one specific weight can be assigned to accumulated silt in a reservoir since deltaic deposits at the upper end are heavy and flocculent deposits at the lower end are light. Also as sediment accumulates and its depth increases, they become compacted. Exposure in reservoirs where the water level fluctuates helps the compaction and shrinkage.

For example, samples taken in the Elephant Butts reservoir in 1915-1916 showed that the unit weight (dry) of sediment varied from about 1.45 t/m³ (90 lb/cu ft) in the delta area to half of that value just above the dam. Further investigation in 1947 showed that the average weight was 1.05 t/m³ (65.9 lb/cu ft). The ultimate average weight of the deposits is estimated to be between 1.2 and 1.34 t/m³ (75 and 83 lb/cu ft). Measurements taken at various depths within the marl sediment in Lake Niedersoutholfen[1] in Germany showed that the unit weight of sediment at the surface of the bed was only 0.346 t/m³ (21.6 lb/cu ft) and increased to 0.72 t/m³ (45.1 lb/cu ft) at a depth of 5 m, 1.18 t/m³ (74.0 lb/cu ft) at 10 m, and 1.43 t/m³ (89.6 lb/cu ft) at 20 m. Investigations of the deposits in the Medina Reservoir[2] indicated that average unit weight was 0.49 t/m³ (30 lb/cu ft) after 13 years of operation, but 5 years later, owing to exposure to the sun and atmosphere in this period, had increased to 1.01 t/m³ (63.4 lb/cu ft).

Among the various factors influencing the unit weight of sediment deposits, the most important one is perhaps the consolidation or shrinkage of the deposits as a result of exposure to sun and atmosphere which is governed by the methods of reservoir operation. Consolidation is again dependent to a large extent on the size composition of the deposit. Clay and fine silt are readily subject to swelling and shrinking while gravel and pebbles will tend to maintain a relatively constant unit weight. Figure 27, reproduced from an excellent report on this subject by E. W. Lane and V. A. Koelzer,[3] shows the effect of proportion of sand on the unit weight of deposit of different reservoirs.

For practical use in designing reservoirs, Lane and Koelzer recommended values of unit weight for various kinds of materials given in tables 11 and 12. The unit weight is assumed to increase with time logarithmically following equation (79).

1. Wissenschaftliche Veroeffenlichung der D.W. Oest.A.V., "Untersuchungen ueber den Niedersoutholfener See in Bayerischen Allgaeu". Innsbruck, 1930.
2. Nickle H.G., Discussion on "The Silt Problem" by J.C. Stevens. *Transactions A.S.C.E.*, vol. 101, 1936.
3. "Density of Sediments Deposited in Reservoirs", Report No. 9 in *A study of Methods used in Measurement and Analysis of Sediment Loads in Streams*, published by the St. Paul District Engineer, US Army Corps of Engineer, St. Paul, Minnesota (USA).

$$\left.\begin{aligned} W &= W_o + \frac{K}{62.4}\log_{10} T \text{ in t/m}^3 \\ W &= W_o + K\log_{10} T \text{ in lb/cu ft} \end{aligned}\right\} \quad (79)$$

where W_o is the initial unit weight (considered at the end of one year), W the unit weight after T years of compaction. The average unit weight over the period (T-1) years as given by Miller[4] is given in equation (80)

$$\left.\begin{aligned} W_{av} &= W_o + \frac{0.4343}{62.4} K \left[\frac{T}{T-1} (\log_e T) - 1 \right] \text{ in tons/m}^3 \\ W_{av} &= W_o + 0.4343\, K \left[\frac{T}{T-1} (\log_e T) - 1 \right] \text{ in lbs/ft}^3 \end{aligned}\right\} \quad (80)$$

According to Miller, it has been found by comparison of actual measured densities with computed values that the values of W_o and K suggested by Lane and Koelzer are satisfactory when the material is predominantly sand. However, when the clay sized material predominates, the answer obtained is too high. As far as clayey material is concerned, a set of W_o values worked out by Miller according to Trask seems to be more applicable. These values are also given in table 12.

For sediments composed of mixtures of various classes of material, the unit weight and K of each class should be combined in proportion to their respective portion of the total sediment. An example of the computation is given below.

A reservoir will always be submerged. Size analysis of sediment consists of 12% sand, 42% silt and 46% sand. The initial unit weight of this material, using the values of Lane and Koelzer since the material is predominantly sand, would be

$$W_o = 0.12 \times 0.48 + 0.42 \times 1.04 + 0.46 \times 1.49 = 1.18 \text{ t/m}^3$$

or

$$0.12 \times 30 + 0.42 \times 65 + 0.46 \times 93 = 73.7 \text{ lb/cu ft}$$

The unit weight of deposit at the end of 50 years will be:

$$K = 0.12 \times 16 + 0.42 \times 5.7 + 0.46 \times 0 = 4.31$$

$$W_{50} = 1.18 + \frac{4.31}{62.4}\log_{10} 50 = 1.30 \text{ t/m}^3$$

$$= 73.7 + 4.31 \log_{10} 50 = 81.0 \text{ lb/cu ft}$$

Average unit weight over the period of 1 to 50 years will be

$$W_{av} = 1.18 + \frac{0.4343}{62.4} \times 4.31 \left(\frac{50}{50-1}\log_e 50 - 1 \right)$$

$$= 1.27 \text{ t/m}^3$$

$$= 73.7 + 0.4343 \times 4.31 \left(\frac{50}{50-1}\log_e 50 - 1 \right)$$

$$= 79.3 \text{ lb/cu ft}$$

4. Miller C.R., "Determination of the Unit Weight of sediment for us in Sediment volume computation. US Bureau of Reclamation, February 1953.

Table 11

Unit weight of deposits for use in design (after Lane and Koelzer)

Reservoir operation	Bourder $d > 256$			Gravel $256 > d > 8$			Coarse sand $8 > d > 1$			Sand $1.0 > d > 0.05$			Silt $0.05 > d > 0.005$			Clay $d < 0.005$ mm		
	Wo		K	Wo		K	Wo		K	Wo		K	Wo		K	Wo		K
	t/m^3	ft/cu ft		t/m^3	ft/cu ft		t/m^3	ft/cu ft		t/m^3	ft/cu ft		t/m^3	lb/cu ft		t/m^3	lb/cu ft	
(a) Sediment always submerged or nearly submerged	2.24	140	0	1.76-2.24	110-140	0	1.49-1.76	93-110	0	1.49	93	0	1.04	65	5.7	0.48	30	16.0
(b) Normally a moderate reservoir draw down	2.24	140	0	1.76-2.24	110-140	0	1.49-1.76	93-110	0	1.49	93	0	1.18	74	2.7	0.74	46	10.7
(c) Normally considerable reservoir draw down	2.24	140	0	1.76-2.24	110-140	0	1.49-1.76	93-110	0	1.49	93	0	1.27	79	1.0	0.96	60	6.0
(d) Reservoir normally empty	2.24	140	0	1.76-2.24	110-140	0	1.49-1.76	93-110	0	1.49	93	0	1.31	82	0.0	1.25	78	0

Table 12

Unit weight of deposits after Trask and Miller

Reservoir operation	Bourder $d > 256$			Gravel $256 > d > 8$			Coarse sand $8 > d > 1$			Sand $1.0 > d > 0.05$			Silt $0.05 > d > 0.005$			Clay $d < 0.005$ mm		
	Wo		K	Wo		K	Wo		K	Wo		K	Wo		K	Wo		K
	t/m^3	lb/cu ft		t/m^3	lb/cu ft		t/m^3	lb/cu ft		t/m^3	lb/cu ft		t/m^3	lb/cu ft		t/m^3	ft/cu ft	
(a) Sediment always submerged or nearly submerged										1.41	88	0	1.07	67	5.7	0.21	13	16.0
(b) Normally a moderate reservoir draw down										1.41	88	0	1.22	76*	2.7	—	—	10.7
(c) Normally considerable reservoir draw down										1.41	88	0	1.30	81*	1.0	—	—	6.0
(d) Reservoir normally empty										1.41	88	0	1.35	84*	0	—	—	0

* Estimated

Measurement of sediment deposits

In general, the purpose of a reservoir sediment survey is to measure the accumulated colume of sediment in a reservoir. This information is useful for:

(*a*) Determining the prevailing and probable future sedimentation damage in the reservoir;

(*b*) Periodically correcting the capacity curve to assure more efficient operation;

(*c*) In combination with similar data from other reservoirs and suspended load measurements, to evaluate the effects of watershed and climatic factors on the rate of sedimentation;

(*d*) Preparing regional sediment production indices for developing design data.

The volume of sediment deposited in a reservoir may be determined by one of the two methods:

(*a*) By determining the present capacity and assuming that the difference between it and the original capacity is a measure of the volume of sediment deposited;

(*b*) By direct measurement of the volume (or thickness) of sediment deposited.

The first method is satisfactory where an accurate map of the original reservoir basin is available. The present capacity is determined by measuring the present water depths at representative points in the reservoir and computing the volume in respect of the surface area. Water depths may be measured by lead-line sounding, by the use of a sounding rod or pole, or by echo sounding.

Where no maps are available or these are of questionable accuracy, it becomes necessary to determine the volume of sediment in the reservoir by direct measurement of its thickness. This may be done by spud, sounding pole or auger. A spud is an instrument developed by the United States Soil Conservation Service specifically for this purpose and is described by Gottschalk.[1]

REMEDIAL MEASURES FOR CONTROL OF SEDIMENT

Reservoir sites are national resources as they are limited in number. An empire comes into being below each large reservoir which supplies water for irrigation and generation of power and sometimes for navigation; provides storage for flood control and water for industrial and domestic water supply and other purposes. As stated before, every reservoir must ultimately silt up, but the life of each reservoir can be considerably prolonged if proper remedial measures are adopted. Brown[2] classifies all methods proposed or tried for the control of reservoir silting into six groups, namely:

1. Selection of reservoir site
2. Design of reservoir
3. Control of sediment inflow
4. Control of sediment deposition
5. Removal of sediment deposits
6. Watershed erosion control

Selection of reservoir site

Most of the reservoirs surveyed in various countries of the world trap from 70 per cent to nearly 100 per cent of the sediment delivered to them by the inflowing streams. The possibility of chosing alternate reservoir sites for water storage should be fully investigated, particularly in the case of smaller reservoirs. If a reliable estimate is made of the prospective rate of reduction of the useful life of a reservoir by silting, it may be found that the construction of a lake at an alternative site, even though more costly, may be ultimately cheaper, if its life is much longer. Besides considering the present sediment production from the alternate drainage areas, the possibilities of utilizing other methods of sediment control should also be considered.

Design of reservoir

Two elements affecting the rate of silting need special consideration in designing a reservoir. One is the ratio of storage capacity to inflow and the other is the design of the water and sediment release openings on outlet works in the dam.

(*a*) *Ratio of storage capacity to inflow*

Study of reservoir silting has shown that one of the most important factors governing the rate of silting or storage loss is the ratio between the original storage capacity of the reservoir and the inflow of water from the drainage basin. The data available in various countries regarding rate of silting of reservoirs per annum per unit drainage area and the ratio of their original capacity to the total runoff is plotted in figure 28. It will be seen that starting

1. Gottschalk L.C., "Measurement of Sedimentation in small Reservoirs", *Proceedings of the A.S.C.E.*, January 1951, pp. 4-6.
2. Brown, C.B., "The Control of Reservoir Silting", US Department of Agriculture Miscellaneous publication No. 521, US Government Printing Office, Washington, D.C., 1944.

from zero, the rate of silting rises rapidly from zero to about 80 per cent as the ratio of capacity to runoff increases from zero to about 1. Thereafter the rate of silting increases rather slowly depending on the characteristics of the different drainage areas. In general, higher rates of silting are noticed where the drainage area lies in regions of smaller and more variable runoff, where the length and shape of the reservoir tend to increase the detention time of inflow, where the sediment load is mainly coarse or highly coagulated, or where outlets and operation practices are such as to release little water from the bottom of the dam and to hold back and store most of the flood flows. Thus constructing a new dam so that it can be raised subsequently may be considered so as to adjust the capacity: runoff ratio after some capacity has ben lost by sedimentation. In practice dams are often raised to impound additional water to meet increased requirements, but there are few instances of dams having been raised to offset sedimentation losses. It may, however, be cheaper to build a substantially lower dam now, and at intervals to raise it until its ultimate height will have given an original capacity in such adjustment to sediment flow that a long useful life results. Similarly, when evaporation is a critical factor, it might be more desirable to maintain a lesser storage capacity by periodically raising the dam than to develop a large excess capacity initially. For example, in connexion with the plans for constructing San Carlos reservoir on the Gila river (USA), it was suggested that increasing the reservoir capacity by raising the dam at a later stage was the cheapest and easiest way of handling the silt problem.

The economic study of proposed reservoirs should, therefore, compare the cost of building a dam of such a type and strength that it can be raised at a later date with the cost of building dams of other types to impound the ultimate capacity today and with the cost of other methods of sediment control.

(*b*) *Outlet works*

Adequate outlet works should be provided in the design of a dam for the release of water and sediment. They may consist of gates on a permanent spillway, pipes or conduits running through the dam controlled by gates or valves, or sluices at various elevations controlled by gates. The potentialities of the various methods for passing sediment through or over a dam wherever reservoir silting is a problem.

Service outlets in dams are generally designed to release about 1 to 2 per cent of the reservoir's maximum storage in one day.. Somewhat greater discharge capacity may be required to vent "density currents" which will be discussed in a subsequent section. Much larger outlets are, however, needed for sluicing and their total discharging capacity should generally range from half to all of the maximum daily discharge.

Control of sediment inflow

Four principal methods have been used to control the inflow of sediment into storage reservoirs:

(*a*) The construction of settling basins
(*b*) The propagation of vegetation screens
(*c*) The location of reservoirs off the main channel
(*d*) The construction of bypassing channels or conduits.

(*a*) *Settling basins*

Settling basins generally cost more per unit of storage than the reservoirs that they protect. Ordinarily a much larger volume of sediment storage could be obtained at a lower cost by raising the height of the dam by a few feet. There are, however, exceptions and the feasibility of providing settling basins should not be overlooked in planning the protection of a new reservoir. Settling basins have the additional advantage that suitable sites for their construction are more easily available particularly in the lower reaches of a river.

(*b*) *Vegetation screens*

These are unquestionably effective in reducing the rate of silting if flood waters are forced to pass through them before entering the reservoir.

(*c*) *Off-channel reservoirs*

The construction of off-channel reservoirs should be carefully considered if sites providing requisite storage are available in the valleys or on side drainages within a short distance of the stream from which the water supply is to be obtained and if it is not necessary to store the entire flow of the stream. An example is the Parakarama Samudra (Tank) in Ceylon which is fed through the Ela Hera canal. The sediment entering the canal, and consequently the tank, can be considerably reduced by means of sediment excluders and extractors or other measures and a reduction of 50 per cent in the rate of sedimentation can be reasonably expected. Where the ratio of the storage required to the total runoff is small, the reduction in sedimentation rate may be even 90 per cent or more.

(*d*) *By-pass channels or conduits*

These include those schemes that involve the diversion of sediment laden flows through canals leading around, or through pipes laid under a reservoir located on a main stream channel.

The various methods require the construction of some type of diversion dam or weir at the head of the reservoir. According to Schoklitsch, such arrangements can be considered provided the reservoir is short and the locality permits construction of such a device at reasonable expense.

Control of sediment deposition

The deposition of sediment in a reservoir can be controlled to a certain extent by designing and operating gates or other outlets in the dam in such a manner as to permit selective withdrawal of water having sediment content higher than the average. The suspended sediment content of part or all of the water in a reservoir is higher during and immediately after a flood, as it takes time for the sediment to settle down. If more water can be escaped at such times, the percentage of total sediment that will settle in the reservoir will be comparatively less. Moreover, water at different levels in a reservoir may contain different concentrations of sediment, particularly during and after a flood. Therefore, if all the water could be escaped at those levels where the concentration of suspended sediment is the highest, it may be possible to discharge a considerable amount of the sediment without its setting in the reservoir.

(*a*) *Density currents*

The tendency of cold rivers to underflow after entering warmer lakes attracted the attention in 1880's of a group of Swiss scientists who noticed that the cold and turbid waters of the Rhine and Rhone rivers appeared to plunge beneath the comparatively warm and clear waters of Lake Constance and Lake Geneva, where they continue as distinct submerged streams.[1]

The stratification or layering of water impounded in lakes and reservoirs is widely recognized as a common natural phenomenon. In its most familiar form it is produced by differences in temperature that cause corresponding differences in density. Stratification that results when dissolved or suspended material produces the essential differences in density is less familiar, but of much greater economic importance. A particular reservoir may contain, simultaneously, layers that are warm, cold, fresh, more or less saline, clear or extremely muddy.

Such complex stratification is, in fact, the usual condition at Lake Mead, where the delightful temperature of the surface layer attracts bathers in summer and the coolness of the temperature of the strata at the depths from which water is withdrawn makes it possible for trout to thrive below Boulder Dam. The transformers, the generator bearings, and in summer the power house offices, are cooled by circulating lake water. Some of the coldest water, however, is slightly brackisk and forms a distinct stratum beneath which is a submerged and very muddy lake.

Such a submerged muddy lake is possible because water strata, unlike rock strata, are mobile. The densest water naturally sinks to the bottom where, if there exists a slope, it may continue to flow until its progress is checked, perhaps by a dam or some other obstacle. A moving stream of this kind is called a "density current", because its slightly greater density gives it the power of motion.

At the very outset, it seems desirable to state and understand what is meant by the term "density current", and to point out that it is used interchangeably with either "stratified flow" or "density flow". When the door is opened between a cold room and a warm one a draft of cold air immediately begins to flow along the floor of the warm room. That draft is a density current. Other factors being equal, cold air is heavier (has a greater density) than warm air, and consequently it flows into the warm room in response to the greater force exerted upon it by gravity. At the same time warm air flows through the upper portion of the doorway into the cold room, as a puff of smoke or a candle flame clearly will demonstrate.

Similarly, iced coffee is denser than that which is hot, and this explains why cream from the same pitcher floats on the one but flows beneath the other. In either case the cream forms a density current, which may be defined rather broadly as a gravity flow of a fluid through, under, or over a fluid of approximately equal density.

A river flowing under air is a gravity flow, but it is not a density current because the density of water is approximately 800 times that of air. One fluid is flowing under another, but the two are not of approximately equal density and, therefore, by definition, surface streams are not density currents. If, however, a cold river enters a warm lake it may continue to flow as a separate stream to the lowest part of the basin. If, for instance, the temperature of the lake is 67°F, and that of the river 60°F, the river water will be approximately 1.0008 times as heavy as the lake water, and the terms of the definition will be fulfilled. The difference in the densities of the two masses of water is at least 10 times as great as has been found necessary to assure stratified flow in experiments conducted in the cooperative laboratory of the Soil Conservation Service at the California Institute of Technology.

These experiments have been made, for the most part, in a glass-walled flume 5 ft (1.5 m) long, 10 in (25 cm) deep, and 2.25 in (5.72 cm) wide. Two large flumes have been used occa-

1. Bell, Hugh Stevens, "Stratified Flow in Reservoirs and its use in Prevention of Silting", US Department of Agriculture, Miscellaneous Publication No. 491, September 1942.

sionally; one is 10 ft (3 m) long and 10 in (25.4 cm) square; the other is 15 ft (4.5 m) long, and 34 in (85.4 cm) wide. Underflows having effective densities (weight per unit volume of underflowing liquid divided by weight per unit volume of overlying liquid) from 1.0001 to 1.0080 having been investigated. Bottom slopes of about 0.25 to 8.0 per cent have been used, and temperatures, carefully controlled, have ranged from 45°F to 95°F. Mean velocities of the currents only rarely have exceeded 4 inches (10.2 cm) per second. Density differences have been obtained ordinarily by the use of fine sediments in suspension, although sugar, sodium chloride, and sodium thiosulphate have been used occasionally, as have differences in temperature.

Density currents may be differentiated as interflows, underflows and overflows. From the standpoint of sediment deposits, the most important type of density current is the underflow type, which because of greater density flows down along the bottom of the reservoir. At the spot where muddy river plunges abruptly beneath the clear reservoir, a mass of debris often accumulates to form a floating island or occasionally, even may become a barrier that reaches from shore to shore. The driftwood and other similar material brought down by the river is stopped at this point by the upstream surface current in the reservoir, and the debris brought by the reservoir current is stopped by the inflowing river. It is as if each is refusing obstinately to accept what is offered by the other. In June 1941 a barrier formed in this way completely blocked a canyon section of Lake Mead at a point between 85 and 90 mi above Boulder Dam and forced upbound boats to turn back. This is a surface indication of an underflow.

Density current overflows commonly occur when ordinary streams enter bodies of saltwater or where water streams discharge into comparatively cold lakes. Unless the overflow water is turbid the phenomenon is invisible. Air travellers who fly along sea coasts frequently catch sight of semi circular areas of muddy water, spreading in the surface of the blue or green ocean like huge tawny fans at the mouth of every flooding stream. Clouds provide the most familiar example of an interflow.

Until comparatively recent years density currents were thought to be rare and mysterious phenomena. Today they are known to be common, although, very frequently, they are invisible or else are found in places where direct observation either is difficult or impossible. This is particularly true of the stratified flows that occur in lakes and reservoirs. If their origin is traced to dissolved salts or to differences in temperature, they are cloaked in invisibility; if they spring from muddy suspensions, they usually plunge beneath the surface and flow unobserved.

In spite of such difficulties field technicians can learn much about them with electric thermometers, conductance cells, current meters, and water samplers. Investigation of this kind are of great scientific value, but they tell very little about the form and appearance of density currents. Behind the transparent walls of an experimental-laboratory flume all such currents can be created and made visible. At the present time, meteorologists of the comparatively new "air mass" school and few specialists in the field of fluid mechanics form about the only group familiar with the behaviour of density currents.

(*b*) *The movement of sediment in relation to density currents*

Many submerged lakes of turbid water trace their origins to very muddy rivers. It has been known for a long time that, because their velocities are greatly reduced and they are no longer able to hold the coarser particles in suspension, sediment-laden streams tend to build deltas when they enter a body of still water. Ordinarily too, the farther a river penetrates a lake, the greater is the reduction in its velocity, so that, eventually, it is unable to hold even the finest sediment in suspension.

This explains why deltas are composed chiefly of particles that are coarse as compared with those deposited on the adjacent floor of the lake and why deposits tend to grow finer with distance from the inlet,[1] but it fails to explain how submerged muddy lakes are related to sediment laden stream.

Generally large reservoirs are created to serve multiple purposes. Their function may be to supply water for irrigation, power production, industrial and domestic purposes. They may also be a vital part in a system of river regulation or flood control, valuable recreation center, and a refuge for wildlife.

Stratification affects all of these interests, and yet it is often impossible to give them any special consideration, because the outlets of many reservoirs have been constructed as if the impounded water were all of one quality and as if only the quantity should be controlled. Given a versatile outlet system and an inventory of the strata within a reservoir an operator who has familiarized himself with the behaviour of density currents, that is to say of layers in motion, can do much towards providing consumers with the quality as well as the quantity of water they desire.

It is becoming increasingly evident that a reservoir is far more than a large water container with an adjustable outlet. It is a machine, a hugh mechine upon which an exceedingly complex civilization is growing steadily

1. This phenomenon has recently been verified by observation in the Krishnarajasagar when this reservoir almost completely dried because of drought.

more dependent for food, drink, power, protection from floods, and even for recreation. Its most efficient operation may depend largely upon a knowledge of the behaviour of density currents and upon the operator's ability to make use of them. By skillful planning he may be able to conserve a part of the storage capacity that normally would be destroyed by the deposition of fine sediments. It is entirely possible that he could supply water of a kind that would seal new canals, destroy weeds in old ones, or even improve the soil of sandy farms.

Density currents of the underflow type are of the greatect interest to the hydraulic engineer because of the possibility that they may be discharged through the dam and thus slow down the rate at which the reservoirs will be filled with sediment. Such currents in Lake Mead have transported sediment over a hundred miles, moving below the level surface of the reservoir. The conditions favourable to density currents of this type are: *(1)* high sediment concentration, *(2)* fine sediment, *(3)* steep stream slopes, and *(4)* large depths of flow, caused by large discharges or narrow channels.

The higher the sediment concentration the greater is the gravity force propelling the current, and the greater the probability of the water settling to the bottom and flowing the entire length of the reservoir. The finer the sediment, the easier it is kept in suspension and the greater the distance it will flow without deposit. The steeper the stream slope the greater the propelling force and the greater the velocity produced, with resultant increases in the proportion of the sediment transported without deposit. The greater the depth of flow, the greater the hydraulic radius of the flow and the higher the velocity.

Whether it is practicable to discharge density currents from a reservoir as a mans of decreasing the rate of reservoir filling is a complex question, depending on the four relationships mentioned above and, in addition, on the position of the outlets and the relative temperature and dissolved solid content of the inflowing and reservoir waters. Other factors to be considered are the damage which might result to downstream irrigation or domestic water installations from the dense flows of fine sediment which would be discharged and the volume of water that can be permitted to be released without depleting the storage of water required for irrigation, power and other uses.

There are indications in both Lake Meade and Elephant Butte Reservoir that in the early stages of reservoir filling, the density currents flowed in the channel of the original river, but as filling progressed the width of the channel increased since the flow could spread out further. As the bottom formed by such currents is very level across the stream, the wider the channel the shallower is the flow. The shallow flow tends to produce lower velocities of flow and causes greater deposits of sediment; consequently less sediment reaches the dam. Moreover, there is likely to be more mixing of the density current with the water in the reservoir as a result of the smaller depth and the lower velocities. Sufficient experience has not yet been accumulated to know how important a factor this mixing action is. Knapp[1] pointed out that there is intense mixing at the point of entrance, where large amounts of energy are made available through the shock losses involved in the change of momentum. However, the loss of energy as a stream enters a reservoir is not generally sufficient to prevent the formation of a density current. Once the current has formed, only mixing at the "interface" can disperse it. Observations verify the hardiness of density currents both in the laboratory and in natural lakes and reservoirs. The experience regarding density currents was summed up at the United Nations Technical Conference on Flood Control by McClellan of the United States Bureau of Reclamation as follows:

> "Experience in the United States regarding the removal of sediment from reservoirs by sluice gates placed near the bottom of a dam has not been too hopeful. Studies on the filling of the Imperial Reservoir on the Colorado river showed that the trap efficiency remained above 95 per cent until the average detention period fell to less than approximately one day. Sediment passing through reservoirs, except the small types fitted with sluice gates, usually moves as a density current, or a diffused colloidal suspension. In most reservoirs the amount is small. Even at Elephant Butte Reservoir on the Rio Grande, where density currents attracted attention, the reservoir is estimated to have trapped 98 per cent of the incoming load. However, if outlets could draw much water from a range of elevations in the reservoir, the evacuation of fine-sized sediments from the reservoir could be improved. The evacuation of sediments from reservoirs, while a subject of some promise, should not be depended on when making an allocation for sediment accumulation. In most cases the trap efficiency should be considered to be at least 95 per cent and rarely should it fall as low as 90 per cent. The exception is found where the reservoir is designed to pass the greater portion of an annual flood with little or no storage such as Aswan on the Nile, or in flood control reservoirs where the reservoir is rapidly evacuated after temporary storage of flood waters."[2]

1. Knapp, Robert T., "Density Currents—Their Mixing Characteristics and their Effect on the Turbulence Structure of the Associated Flow", *Proceedings of the 2nd Hydraulics Conference, University of Iowa Studies in Engineering, Bulletin 27*, 1943.
2. ECAFE, *Proceedings of the Regional Technical Conference on Flood Control in Asia and the Far East* (Flood Control Series, No. 3), p. 23.

Removal of sediment deposits

The sediment deposits can be removed by excavation, dredging, draining, flushing, flood sluicing or sluicing with controlled water and sluicing with hydraulic and mechanical agitation.

Excavation is economically feasible only under very special circumstances. Where the excavated material can be sold for commercial sand or gravel, the cost, if any, to the reservoir owner may be small enough to justify such an operation. For instance, in the case of the Damodar Valley Dams, the sand to be removed is expected to be used in "sand stowing" of coal mines that abound in the area. The sediment deposits could also be used for spreading on eroded or inferior soils to make them more productive or for strengthening embankments in the vicinity. One of the critical problems involved in excavation, where the material cannot be used otherwise, is the disposal of the material and the cost of land required for its disposal.

Dredging is expensive as it needs the purchase and maintenance of expensive equipment. Dredging is generally undertaken for the maintenance of a lake at a particular site rather than for maintaining the storage capacity of the reservoir. The concensus of opinion is that dredging is not economically feasible for most water storage projects and its resorted to mainly on small reservoirs for domestic water supply. The costs involved may, however, become justified as sites become scarce and the value of their services increases.

Draining and flushing involve the relatively slow release of all stored water in a reservoir through gates or valves located near the bottom of the dam, and the maintenance thereafter of open outlets for a shorter or longer period during which normal flow cuts into the sediment deposit. This is similar to the flushing practiced on rivers in India to wash out the accumulated sediment upstream of a weir or barrage. On large reservoirs, this can be practiced only when it is possible to release all the stored water. There is no difficulty in doing so on reservoirs meant for flood control only or for flood control and irrigation, because at certain seasons of the year water may not be needed for irrigation, particularly during the rainy season. Even though it may not be possible to release the water every year, periodically a run-off season will have such characteristics that water can be completely drained from the reservoir and the gates kept open for a period of weeks or even months. However, the difficulty arises with reservoirs used for generation of power, unless alternative sources of power can be provided. Moreover it is not certain if this method would be effective on large reservoirs. Hepmhill[1] concludes "The stream would cut only a narrow channel through the deposits, and the scouring action would cease before any considerable proportion of the deposit had passed out of the reservoir. The practice would be improved by directing the flow of water against the banks."

1. Hemphill, R.G., "Silting and Life of Southwestern Reservoirs", *Transactions of the A.S.C.E.*, vol. 95, pp. 1060-1072.

Brown[1] states "The benefits that may be obtained during any reasonable length of time, ranging from days to weeks, would appear to consist of removing 1 per cent or less of the sediment in most basin type reservoirs, 3 to 10 per cent in moderate sized channel type reservoirs and 25 per cent or more in small channel type reservoirs if a large discharge is available during the period of flushing. On the other hand, compaction will be most effective in basin type reservoirs and may reduce the silt volume eroded to 1 per cent or less in a few days, several per cent in a few weeks or months and up to 50 per cent in the course of several years exposure.. Draining and flushing the majority of reservoirs is a simple and inexpensive practice if the water loss can be afforded but its benefits are often proportional to the low cost."

Flood-sluicing is the operation of removing sediment deposits from a reservoir through large sluices near the base of a dam by utilizing the scouring action resulting from the sudden release of the stored water or of flood flows entering the reservoir, without materially lowering the water level. Experience shows that the effect of such releases does not extend for beyond the gates. However, they are effective in as much as the flood flow may pass through without depositing its sediment load in the reservoir. Their effect is somewhat different from that of flushing with the reservoir level drained down, as lowering of the reservoir level provides very steep slopes and consequently high velocities throughout the length of the lake which during large flood flows scours the bed as well as erodes the sides and fairly large quantities of silt can be washed out if the process is repeated every year before the silt has time to get compacted.

Sluicing with hydraulic or mechanical agitators for stirring up the silt deposits along with the flushing may be useful, but does not seem to have been tried.

Watershed erosion control

This includes all those measures which are effective in preventing or delaying the movement of soil and rock particles from their point of origin in the drainage area to the reservoir.

Fundamentally, erosion control differs from all other methods of silt control in as much as it aims at dealing with the problem at its source.

1. Brown, C.B., "The Control of Reservoir Silting", US Department of Agriculture, Miscellaneous Publication No. 521, US Government Printing Office, Washington, D.C.. 1944.

The scope of erosion control is as broad as the fields of soil conservation, forestry and land management. It has been shown in the section dealing with soil erosion that soil conservation and land management are effective in reducing the rate of erosion to a considerable extent. The actual extent of this effectiveness will naturally depend on the extent of erosion in different parts of the basin and the steps taken to prevent it.

It is of the greatest importance to determine accurately or even approximately the various sources of silt within the basis of a reservoir before steps can be taken to deal with it. The contribution of sediment by different catchments and sub-catchments due to the variation in soil characteristics, plant cover, slopes, physiographic variations and other characteristics is so different that a complete survey of the whole catchment, if possible, including its various sub-catchments or at least of those parts of it which contribute most of the sediment, is essential in order to find out the sources of silt entering a reservoir.

Once the sources of sediment have been located it is not so difficult to devise measures to control the sediment at its source. The work involved is, however, vast and sometimes very expensive and the best that can be done is to deal with the various sources in order of their importance. For example it is no use proceeding with forestation and land management if gullies contribute 50 per cent of the sediment and stream channel erosion contributes another 30 per cent.

The method that contribute to erosion control are many and varied and are outlined by Brown as below:

(*a*) Afforestation
(*b*) Regrassing and grassland management
(*c*) Cultivation practices
(*d*) Protected channelways
(*e*) Gully control
(*f*) Ponds
(*g*) Highway and railway erosion control
(*h*) Stream bank erosion control
(*i*) Flood plain protection
(*j*) Water spreading
(*k*) Reservoir shore line protection, and
(*l*) Wild life plantings.

Brown concludes "The greatest claim that can be made, or needs to be made, for the method of watershed control, is that it goes at the root of the problem, aiming to eliminate the cause of sedimentation. It is primarily the surgery that removes the cancer, by comparison with the palliative ointments that may keep the patient on his feet a few years more."

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Chapter V

ACTION OF SEDIMENT ON THE REGIME OF RIVERS[1]

Many factors influence the regime of rivers, such as the topographical difference of elevation from river source to river mouth, the lining material of the bed and sides of the river and the quantity as well as seasonal variation of water and sediment discharge. With all these factors exercising their influences, the regime of rivers tend to attain a state of equilibrium in which the river slope, river course, and cross-section will fit the condition of water and sediment flow. Among all these factors, sediment flow is of paramount importance, particularly for stretches of rivers flowing through alluvium, for the lining material of alluvial rivers is derived from the deposits of sediment flow, which, together with the sediment load and water discharge, determine the slope, cross-section and the course of rivers.

The equilibrium of river regime is sometimes considered from the point of equilibrium of energy. The available energy of a stream flowing on an inclined plane is balanced by the dissipation of energy due to the bed and side friction, internal friction as well as the energy required to transport the sediment load. The excessive energy of high water flow, as compared with low water, is consumed by the transportation of sediment, which varies in general with the discharge. Any change of sediment charge will result in change of river regime, such as aggradation due to excessive sediment load and degradation below a dam due to trapping of sediment in the reservoir. Without changing the sediment charge, a change of flow pattern, on which the transportation capacity of sediment depends, will also change the river regime, such as the retardation of flow by the construction of a weir across the river, the withdrawal of flow into irrigation canals, and the branching of rivers. If the river has not yet attained a condition of equilibrium and is building up the required slope to transport the sediment load, as on the Yellow River, aggradation will take place. Changes in river regime may also be due to other causes such as the extension of river delta resulting from sediment deposition. The major effects of sediment on river régime are briefly discussed below.

AGGRADATION DUE TO EXCESSIVE SEDIMENT LOAD

Sediment load in excess of what the river normally carries will disturb the river regime. Observations made on some rivers in South Korea and Java indicate that deforestation during the Second World War had increased the sediment load and deteriorated the river bed. In connection with excessive sediment load, the Yellow River presents the most interesting case. The lower course of this river, having not yet attained the stage to transport all its sediment to the sea, is subject to aggradation. The average silt content of the Yellow River at Shenhsien is 4.4 per cent by weight and the recorded maximum during flood reaches 46.14 per cent. The annual silt run-off from the headwater region to the lower course on the alluvial plain is 1,890 million tons or roughly 1,260 million m^3 of which 75 per cent is discharged during the flood period from July to October. The present dike system, from the river mouth landwards to a distance of 500 km, was constructed in 1,856; and from there onward in a distance of 225 km, to the outlet of a gorge section, the dikes were constructed in 1495. It is quite apparent that dikes were built on level ground and that the surface of land, both within and outside the dikes, was at the same elevation. A cross-sectional survey of river carried out in 1933 indicated that the area within the main dikes (or the high water bed) was heavily silted up, the depth of sediment ranging from 1 to 7.1 m. The result of the survey is given in table 13.[2]

1. See also Bureau of Flood Control, ECAFE, *River Training and Bank Protection*, (Flood Control Series No. 4). December 1953.
2. Chang, S.C., "Quantitative Analysis of Silt Deposition of the Yellow River", *Hydraulic Engineering* (China), Vol. 15, No. 1, 1947 (Nanking, Chinese Society Hydraulic Engineering, 1947) p. 93.

Table 13

Distance from river mouth (km)	Distance between dikes (width of high-water bed) (km)	Average height of river bed above adjacent land outside the dike (m)			Rate of deposition (cm per year)
		Left side low water channel	Right side low water channel	Weighted mean	
0 — 185	1.5 — 2.0				
185 — 303	1.5	3.0		3.0	3.90
303 — 500	6.0 — 14.0	1.0	1.0	1.0	1.30
500 — 583	8.5 — 13.0	7.1	4.3	5.7	1.30
583 — 725	5.0 — 10.0	4.8	3.3	4.5	1.03

From the above table it will be seen that the annual aggradation of river bed above adjacent land is approximately 1.0 to 1.3 cm. The volume of sediment deposited within the diked section of the river per annum, however, constitutes only 6.4 per cent of the total annual sediment load discharged into the lower course of the river. Of the balance, 68.4 per cent was deposited outside the dikes due to breaches and 45.2 per cent was carried to the sea.

DEGRADATION OF RIVER BED DOWNSTREAM OF A WEIR

Retardation of flow upstream of a weir will cause deposition of sediment load, particularly the coarser particles, and the flood flow passing down the weir will be partly relieved of its sediment load. It will then possess excessive energy to degrade the river bed downstream of the weir and will pick up as much solid materials as it can carry. This phenomenon was observed on many rivers in India and Pakistan as early as 1882 and was proved by the fact that the gauge level for a specific discharge decreased after the completion of a weir such as Khanki, Rasul, etc. The same phenomenon was also observed in the United States after the completion of a series of dams on the Colorado river. Data on degradation below some weirs in India and Pakistan and some dams in the United States are given in tables 14 and 15.

Table 14

Degradation of river bed downstream of some weirs in India and Pakistan

River	Chenab	Chenab		Sutlej		Jehlum	
Weir	Khanki	Marala		Rupar		Rasul	
Date of operation	1891	1910		1882		1900	
Degradation (ft)	4.5	steady degradation	1.0	5.0	continued	1.0	5.5
Period	7 yr 1891—98	9 yr 1910—19	5 yr 1922—27	immediately after operation	8 yr 1882—1890	10 yr 1902—1912	7 yr 1912—19
Rate of degradation (ft/yr)	0.64	—	0.20	—	—	0.10	0.79
Recovery (ft)	4.5	1.0	—	—	complete recovery	—	2.0
Period	11 yr 1898—1909	3 yr 1919—22	—	—	27 yr 1890—1917	—	11 yr 1919—1930
Rate of recovery (ft/yr)	0.44	0.33	—	—	—	—	0.18

Table 15

Degradation of river bed downstream of some dams in the United States

River	Colorado			
Dam	Hoover	Davis	Parker	Imperial
Date of operation	1935	—	1938	1938
Distance below Hoover Dam (mi)	0	70	150	305
Volume of degradation downstream of dam (million cu yd)	125	—	135	65
Period	11 yr 1935—46	—	8 yr 1938—46	6 yr 1940—46
Degradation (ft)	at mi downstream 5 at 13 mi downstream 9 at 26 mi downstream 4 increased to 9	—	—	—

If the movable bed material below a dam is deep and uniform in size, the bed may be lowered considerably. If the bed material is composed of sediment of various sizes, a condition generally prevailing in rivers, the clear water released from the dam removes the finer material and leaves a sort of paving on the stream bed which reduces the rate of degradation.

The lowering of the bed level occurs firstly near the dam and gradually extends downstream to a point where the stream has picked up a capacity load as determined by the prevailing slope and the discharge of the regulated flow. Interesting enough, a part of the eroded material may re-deposit some distance further downstream and thus induce aggradation after a stretch of degradation. This can be explained by the fact that the river slope and the regulated discharge from a dam in the river downstream of the stretch of aggradation is not capable of transporting all the sediment picked up by the clear water flow. Such aggradation may cause serious flooding as a result of blocking of the flood flows.

In the case of a weir or barrage, after the channel upstream of a weir is silted up and the total sediment again passes over the weir, a gradual filling of the degraded bed takes place until the original level is resumed, which is shown by the examples given in table 14.

There are many variables affecting the rate as well as the amount of scour which will take place in a river bed after construction of dams on the stream. Therefore, the problem of predicting the exact rate and amount to be expected is a complicated one. A hint of a possible solution is proposed by Stanley[1] on the basis of records of measurements on the Colorado River.

Early in the study of sediment on the Colorado it was found that the suspended load carried by the river varied approximately as the square of the discharge, other variables remaining constant.

Taking the rate of sediment load transported by a discharge of $Q = 15{,}000$ cu ft/sec of water as standard, designated as $Q_{s\ 15{,}000}$, the rate of sediment transported at any other discharge Q_{sq} is converted into the value corresponding to $Q = 15{,}000$ cu ft/sec by the simple relation of $Q_{s\,15{,}000} = Q_{sq}\left(\frac{15{,}000}{Q}\right)^2$. The average annual rate of sediment transported for $Q = 15{,}000$ is then plotted against the calendar year. A typical curve is reproduced in figure 29.

Future rates and volumes of scour could be estimated from this curve with reasonable accuracy if future actual discharges were known. The possibility of applying the method to other streams for the purpose of estimating the scour as suggested by Stanley is interesting. It will be noted that the analysis shows only the rate of scour plotted against time, and therefore the effects of such variables as slope, alignment, and size composition of the river bed are eliminated. If a smooth curve could be maintained by applying corrections to the rate of scour, the corrections to be determined by slope and alignment changes and changes in size composition, the curve should be applicable to other streams.

1. Stanley, J.W., "Effect of Dams on Channel Régime", Proceedings of the Federal Inter-Agency Sedimentation Conference, 1948, p. 165.

An important practical aspect of the effect of degradation is the lowering of river bed and hence the water level, which may endanger the structure of the dam and put the turbines out of action. On the other hand, the extra head that becomes available by degradation is capable of giving extra power if the turbines are originally placed at the proper elevation, to take care of the likely degradation.

The suggestion of utilizing degradation to deepen the Yellow River is of interest. It has been proposed to store the heavily sediment laden flow on the vast foreshore areas within the dikes and release the clear water, after it has deposited its sediment load, to scour the main channel. In this respect, one can expect that a reduction of sediment load by soil conservation measures will have the effect of inducing a deepening of the river channel.

AGGRADATION DUE TO RETARDATION OF FLOW

Retardation of flow by dams causing deposition is discussed in a separate section under "silting of reservoirs."

AGGRADATION DUE TO CHANNEL WITHDRAWAL

Aggradation of river bed occurs when a large portion of clear water is drawn by an irrigation canal leaving the sediment load, particularly the coarse particles, to be carried by the reduced flow. Measurements carried out on the river Indus and its tributaries showed, that the river bed below the barrages, after being restored to the original level after degradation, attained even higher levels due to the reductions of discharge by canal withdrawal.[1]

AGGRADATION OR DEGRADATION DUE TO BRANCHING OF RIVERS

In many instances when rivers are divided into two or more branches, as in the case of deltaic rivers, it is not possible for every branch to maintain a state of equilibrium in respect of sediment flow. If the sediment load is in excess of the transportation capacity of any one of the branches, deposition is bound to occur. The deposition tends to reduce the cross-sectional area which, in turn, decreases the discharge and accelerates further deposition. The other branch, passing more discharge, will be degraded or enlarged.

An example of aggradation of branches is the river Ganga in India.[2] The Ganga branches into Baghirati and Padma. Up to 16th or possibly 17th century, Baghirati received throughout the year the flow from the Ganga. Gradually, the Ganga directed its waters to the east towards the Padma, which has not become by far the more important branch. The Baghirati is at present practically separated from the Ganga for eight months of the year and during flood time now receives an almost negligible fraction of the flow.

Another example is the Yellow River. After the dike breach of 1938, the river divided into two courses, with the new course running southeastward to the Huai river. The old course direct to the sea has gradually silted up and in a period of only four years the old course was entirely dry except during extra-ordinary flood flow. The Yellow River is constantly losing water in its course of flow by seepage through higher-than-ground dikes. This must also contribute as a cause to its aggradation of beds.

Such instances can also be seen on the Yangtze river where the flow branches off into the Tungting Lake.

AGGRADATION OF RIVER BED DUE TO EXTENSION OF DELTA

The rate of advance of coast line for some rivers is given below (it must be noted that such a rate is proved only near the main mouths of the river and not on the coast lines of the whole dalta):

	(per century)
Irrawaddy	5 km (3 mi)
Mekong	5 km (3 mi)
Red River	5–8 km (3–5 mi)
Yellow River	5 km (3 mi)
Mississippi	6–5 km (4 mi)
Po	5 km (3 mi)

It is often suggested that the extension of delta would result in an aggradation of river bed, if the river is to maintain the same slope. However, no detailed data is available to ascertain the quantitative effect of delta extension, as it is noticeable only over long periods of time. Besides, many other factors like ocean currents, prevailing wind direction, etc. influence aggradation and greatly complicate the problem.

1. Foy, T.A.W., 'Régime Level Changes on the Indus System", Punjab Irrigation Branch Paper 16, Class B, 1944.
Nicholson, H.W. and Trench, *Report on the Effect of Bhakra Dam on River Supplies*, Government Printing, Punjab, Lahore, 1930.
2. Majamdar, S.C., *Rivers of the Bengal Delta*, Bengal Government Press, Calcutta, 1941.

SEDIMENT MOVEMENT IN TIDAL RIVERS

General considerations

The flow in tidal rivers is essentially the escillatory movement of two bodies of water of different densities. The heavier sea water drives the fresh water of the river landward during the spring tide; while the stored up fresh water along with the salt water flow down to the sea during the neap tide.. The period for a complete cycle of the landward and seaward movement of the two bodies of water is roughly 12 hours and 25 minutes and the distance covered by the oscillatory motion is dependent on the strength or the amplitude of the tide in comparison with the fresh water discharge of the river. The configuration of the river itself also exercises an important effect on the amplitude and form of the tidal wave as the latter travels upstream.

With respect to sediment movement in tidal rivers, the problem of practical significance is indeed the question of silting and scouring. In order to understand this problem one must know how sediment behaves in tidal rivers. What happens to the sediment carried by the fresh water when it meets the salt water or at the interface of the two bodies of water and then what happens afterwards during the oscillatory movement of the tidal current. In addition to the sediment load of the stream, sediment may be added to the river from other sources such as littoral drift. When material carried by littoral drift reaches a river mouth, it may be picked up by the tidal currents and carried into the river channel by the spring tide and again seaward by the neap tide.

If an analytical approach to the problem is attempted, in regard to silting and scouring, it would be very useful to draw up a balance sheet of sediment movements both in the upstream and the downstream directions by determining the distances travelled by any particle upstream with the spring tide flood current and downstream with the ebb current. The resultant of the distances travelled during one complete tidal period, either in the upstream or in the downstream direction, will give a good indication of silting or scouring. This analysis may be further simplified by considering various factors separately and then combining them together as follows:

(*a*) To investigate sediment movement of a river not subject to tidal influence or only subject to very weak tides where no reversal in the direction of river flow takes place anywhere in the river.

(*b*) To investigate sediment movement, over a tidal period, of a tidal current running into a still water channel (the channel having a certain depth of water but no discharge nor flow). This is to ascertain the net result of sediment movement, after a complete period of oscillation, either upstream or downstream with respect to its initial position.

(*c*) To super-impose a tidal current over the river flow as mentioned under (*a*) and (*b*) above and to investigate the resultant movement of sediment with respect to its initial position; whether the sediment load is from the stream, or from the sea.

It may be pointed out that silting or scouring of tidal rivers depends not only on the resultant movement of the sediment in the river itself as mentioned under (*c*) above, but also on the condition just outside the river mouth. Even when the river flow together with the tidal flow is in a position to transport the entire sediment load, sand bar formation just outside the river mouth may tend to induce deposition inside the river channel. This problem will be discussed later.

Rivers subject to weak tides

For rivers subject to weak tides where no reversal in the direction of flow takes place anywhere within the river, or for rivers discharging into a salt water lake without tides, the sediment movement in the river is essentially fluvial in character and has been discussed in chapter II. In such cases the sediment transportation power of the stream itself governs the silting or scouring of not only non-tidal rivers, but also rivers subject to weak tidal action.[1]

As soon as the river discharges into the sea, the bed load will cease to move and the larger particles of suspended load will also settle down. In case the sea is shallow and there is no littoral current to carry away the sediment, a sand bar will form in front of the river mouth, which action will be helped by the beach forming sea waves.

The river flow carrying the remaining fine or colloidal sediments in suspension, when meeting sea water of heavier density, will spread out over extensive areas in the sea. Colloidal particles will settle very slowly if they are not subject to flocculation. It is well known that colloidal particles bear a charge of electricity which may be either positive or negative according to the nature of the colloid. When two colloidal jets of opposite electric charge are mixed, or when they are in contact with electrolytes of opposite sign in sea water, pre-

1. Wheeler, W.H., *Tidal Rivers*, Longman, Green & Co., p. 123. Wheeler cited the instance of the Trent and Ouse rivers in England where freshets together with ebb tide kept the sediment moving downward to the sea, which remained in the lower reaches during periods of low flow. In the case of a stretch of the Huai river from Tientsin to the sea, which is subject to tidal flow, the river bed deepened whenever there was a dike breach upstream of Tientsin, as the relatively silt-free return flow was capable of picking up more sediment.

cipitation takes place. In such cases, the colloidal particles will settle down much more rapidly. There are many materials and reagents which undergo such a change but it should be remembered that some silts are not subject to this reaction.

It is interesting to compare the case of sediment laden flow running into a fresh water reservoir or lake with that of the same flow running into a body of tideless salt water. While sediment laden flow will dive under the sweet water of a reservoir, it will spread over the heavier salt water.

Rivers subject to strong tides

Rivers subject to strong tides include those rivers where a reversal in the direction of flow takes place within the river. As the fresh water of the river meets a body of intruding salt water or the flood current, the former is headed up by the latter. The salt water, on account of its heavier density, moves upstream in the form of a wedge near the river bottom. Velocity measurements taken at this interface in models[1] have shown in many instances that velocity at the surface as a result of river flow is directed towards the downstream whereas velocity at the bottom as a result of salt water intrusion is directed towards upstream. The greater the density of sea water as compared with that of the sediment laden river flow, the more pronounced will be the wedge formation. In case the sea water outside the river mouth is very much diluted by the river flow, the wedge formation will not be so marked.

Mixing generally takes place along the interface of the salt water wedge with river water. This interface appears to be made up a mixture of salt water and fresh water, as the latter is being discharged downstream or is being impounded by the flood current. This interface is usually not very well defined.

The salt-water wedge moves upstream during the spring tide while the impounded fresh water along with the salt water runs down to the sea during the neap tide. The salt water wedge penetrates farther upstream during low water river flow and is driven downstream during the high water flow.

Considering a specific point in the tidal reach of a river, as the salt water wedge approaches this point from downstream, the river flow, upon meeting this salt water wedge, loses its velocity. Materials carried as bed load will cease to move and the coarse particles of the suspended load will also settle down in accordance with Stoke's law. Very fine or colloidal sediment in suspension will settle down very slowly or may not settle down at all, while particles subject to electro-chemical reaction will flocculate upon contact with the salt water. If the river at the point is subject to spring tides deposit will take place much more rapidly.

As the salt water wedge travels landward, the point under consideration will be subject to a flood current directed upstream. Sediment on the river bed may then be brought into motion as bed and/or suspended load. At the instant when flood flow changes into ebb flow sediment particles cease to move and then resume their downward movement during the ebb flow which is reinforced by fresh water.

When the material carried by litoral drift reaches a river mouth, it is picked up by the tidal current and is carried into the river channel by the flood current while the ebb current carries it back towards the sea. Thus it is subject to the same oscillatory movement as the sediment load of the river.

Regarding the silting and scouring of tidal rivers as pointed out before, a simplified analysis of the sediment transportation power of the tidal current may throw some light on this complicated problem. In doing so, one may start with the case of a tidal wave running into a channel with no discharge but with a certain depth of water and then superimpose the river flow over the tidal flow.

One may either trace the movement of a particle of sediment over one complete tidal period of flood and ebb or sum up the sediment transportation power of the tidal current at a particular point over the same period of time, the latter being much simpler.

As a tidal current is essentially non-steady and non-uniform in character, and as sediments behave quite differently as bed, suspended and colloidal loads, a correct expression to define the sediment transportation power of a tidal current has not yet been determined. As an approximation, it may be assumed that the transportation power is proportional to the square, cube or any higher power of mean velocity of flow (V^m with $m>1$) and that the mean velocity is approximately proportional to the discharge Q.

The transportation power will then be proportional to Q^m, m being greater than unity.

The instantaneous discharge of the tidal current and its sediment transportation power Q^m at any point can be plotted against time over a complete tidal period. Areas under the Q curves give the total quantity of flow while the areas under the Q^m curves give the work done in transporting sediment. The difference in the area enclosed by the Q^m curve for the flood and ebb currents respectively indicates the net result of sediment movement, either upstream or downstream as the case may be (figure 30).

1. Rhodes, R.F., "Effect of Salinity on Current Velocities", US Corps of Engineers, Committee on Tidal Hydraulics, *Report* No. 1, 1950, p. 94.

Considering the case of a tidal current running into a channel of still water, if the Q curves of the flood and ebb currents are symmetrical, the areas under the Q and Q^m curves in both cases will be the same and the net sediment transportation power will be zero. There should be neither silting nor scouring. In actual practice, even with a symmetrical Q curve at the mouth of the channel, a deformation is experienced in the shape of the Q curve as the tidal wave travels upstream. This is due to the fact that the crest of every tidal wave partly overtakes the trough preceeding it. As a result of which, the Q curve at some distance upstream shows a sharper rise in discharge with a corresponding shorter period for the flood flow and a slower outflow and consequently a longer period for the ebb flow, as shown diagrammatically in figure 30 (*b*) Although the areas under the Q curves for the flood and ebb flows are exactly the same, indicating that the same quantity of water is discharged into and out of the channel, the areas under the Q^m curves are, however, different. In such a case, the area under the Q^m curve for the flood current movement is usually greater than that for the ebb current and the net result will be a movement of sediment towards upstream, which indicates deposition.

When a tidal wave runs into a river having a certain discharge Q_o, a symmetrical Q curve will also experience a deformation as mentioned above.. The river discharge will be impounded during the flood period and released during the ebb period. This amounts to increasing the discharge of the ebb current thereby increasing its transportation power in the downstream direction. As pointed out before the flood current has usually a greater sediment carrying power than the ebb flow in a channel without discharge. This excess is counter-acted by the stream flow. The net result is represented by the difference between the areas enclosed by the Q^m curve of the flood current and the $(Q + Q_o)^m$ curve of the ebb current as shown in figure 30 (*c*) which is dependent on the shape of the Q curve as well as on the magnitude of the river flow Q_o.

This effect of river discharge on silting and scouring also explains the common fact that material deposited in a shoal during average flow may be removed during higher stages of river flow.

Tidal flow in natural streams is a very complicated phenomena as both the Q and Q_o curves change from place to place and from time to time. In the training of tidal rivers, the Q curve changes its shape as a result of change of cross-section, alignment and other features of the river. It may be desirable to investigate the effect of regulation works on the Q curves with a view to finding a more systematic solution to the complicated problem.

There are many problems in tidal rivers that can not be tackled in a systematic manner. One complication frequently encountered is that the ebb current may largely follow one channel and the flood curernt another, with a large shoal between the two. This tends to encourage shoaling in the ebb channel during flood flows and in the flood channel during ebb flows. In training tidal rivers in the interest of navigation, flood control or drainage, reliance is nowadays placed on model tests, which have proved to be very useful.

As far as the deposition of river sediment load outside the river mouth is concerned, it is similar to what has been explained under rivers subject to weak tides. As regard the movement of sediment derived from littoral drift and subsequently carried into the river by flood current, it is again a problem of the difference of sediment transportation power of flood and ebb currents together with that of the river flow as already mentioned before. Shoaling will take place inside the river if the transportation power of the flood current is stronger than that of the ebb current combined with river flow, and sand bars will be formed outside the river mouth if the reverse is the case. Over the shoals, the currents usually find their way in multiple, meandering, shifting and shallow channels. One interesting aspect of the shoaling process is that tidal currents, by creating inside shoals or offshore bars, tend to diminish their own transportation power as the magnitude of the Q curve decreases.

There are many problems requiring detail study for a thorough understanding of the sediment movement in tidal rivers such as (1) the mechanics of mixing of two bodies of water (2) the behaviour of bed, suspended and colloidal load during the mixing (3) Agitation of river bed material during mixing (4) Deposition gf suspended or coagulated material and its picking up again when the direction of flow is reversed and (5) criteria to indicate correctly the sediment carrying power of tidal currents.

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Chater VI

SAMPLING AND ANALYSIS OF SEDIMENT[1]

SAMPLING OF SEDIMENT

When dealing with the theory of sediment transport in flowing water, mention has frequently been made of the samples of the sediment either from suspension or from the bed. Assumption has always been that such undisturbed samples of sediment can be obtained from the flowing fluid. This, however, is not always the case. It is comparatively easy to obtain samples of suspended load, but to obtain samples of the bed lod is rather difficult.

Sampling of bed load

In the previous discussions it has been explained that "bed load" may be defined as that part of the sediment load of a stream, which moves on the bed, hopping, rolling or saltating. If the turbulence is high, part of the load goes into suspension and when the turbulence is less the movement of the bed material extends only up to a short distance above the "bed". It will therefore be seen that taking a true representative sample of such a medium is extremely elusive. A grain size analysis of the sample might give an idea about the coarseness of the material, but to assess the magnitude of the bed load that passes any cross-section of the channel at the time of sampling has been a baffling problem. Many people have worked in this field and developed different types of samplers; almost everyone of these samplers has been tried with indifferent success.. The samplers developed in Europe, India, the United States of America and other countries are not greatly diversified, for the most part falling into a few general classifications, each of which includes samplers of the same principles of operation and construction. Most of the types have been developed in Europe, and only in very recent years have American engineers contributed appreciably to the improvement of the equipment for sampling bed-load. The various types of bed-load samplers which are in use in different parts of the world are described in detail in a report entitled "Equipment Used for Sampling Bed Load and Bed Material".[2]

By definition a bed-load sampler must be capable of measuring the rate of transportation of the bed-load material. The rate of transportation of such material as is rolled or pushed along the bottom can be measured only by direct volumetric measurement and can not be conveniently or accurately derived by separate measurements of the flow velocity and sediment concentration, such as are relied upon for suspended load measurement. This is due to the fact that bed load material does not move at the same velocity as the water, and that furthermore, close to the stream bed, both concentration and velocity are changing rapidly with depth and time.

In this bottom layer the rate of transportation can be determined accurately only by means of an apparatus which will trap all solid particles moving through a certain part of the cross section during a measured period. Such is the principle of all true bed-load samplers. It may be pointed out that samplers for the determination of sediment concentration close to the stream-bed are not considered bed-load samplers, in the sense of the definition now used.

(*a*) *Classification of bed-load samplers*

Bed-load samplers, as defined, may be grouped into distinct classes, according to their type of construction and principle of operation. The rate of bed-load movement for all types is determined by placing the sampler on the stream-bed and measuring the amount of material collected in a given time.

1. A very comprehensive study on measurement and analysis of sediment loads in streams was planned and conducted jointly by the Sub-Committee on Sedimentation, Federal Inter-Agency Basin Commitee, USA. For detailed information about this subject, readers are referred to the excellent set of reports in 9 volumes published by the St. Paul District Engineers, St. Paul, Minnesota (USA).

2. Report No. 2 in *A Study of Methods used in Measurement and Analysis of Sediment* Loads in Streams, published by the St. Paul District Engineer US Army Corps of Engineers, St. Paul, Minnesota (USA).

(i) The basket type

Probably the oldest and most common class of bed-load samplers is the box or basket type. Fundamentally this type consists merely of a box or basket, generally made of meshed material, which is lowered to rest on a stream-bed with the upstream end opened to catch a sample of the moving material. The introduction of the sampler into the stream, causes an inward resistance to flow and a resultant lowering of the stream velocity. Hence the entrance velocity is decreased from that of the undisturbed stream, causing some of the material to drop out before entering the basket. Thus, the efficiency, that is, the percentage of the material moving towards the sampler which is actually caught by it, is less than 100 and must be determined to obtain reliable results with this type of equipment.

(ii) The tray or pan type

This type consists of a flat pan or a tray-shaped device with baffles or slots to check the moving material. Since this type also causes obstruction to the stream, and consequent reduction of entrance velocity and movement of material, it must be calibrated to determine its efficiency.

(iii) The pressure-difference type.

This type is designed to overcome the objection of decreased velocity and bed-load movement at the entrance to the sampler. A rational solution to the problem lies in the formation of a pressure drop at the exit of the apparatus just sufficient to overcome the energy losses, thus giving the same entrance velocity as that in the undisturbed stream. This is accomplished by designing the instrument with a section diverging in a downstream direction, which will cause suction at the entrance. With such a diverging section the velocity towards the downstream end of the sampler is decreased and some of the material is deposited. Thus if this section is of sufficient length the necessity of a collecting screen at the exit end may be eliminated. Some of the samplers of this type have a collecting screen, while others have only baffles to check the movement of the material.

(iv) The slot type

Another system which has been used in special cases is to construct slots in the stream bed and allow the moving material to drop into them. In one type the material is piped to the bank from the slots and the rate of bed-load movement determined, while in another type the slot is dug into the bed to collect a sample for size analysis only. A special slot type apparatus had been developed in the Punjab Irrigation Research Institute and extensive studies[1] were made in a small distributary canal. Three slots were used with various dimensions and spacing. Broad conclusions arrived at from these experiments are as follows:

a. Slot No. 1 did not catch all the bed load. Even by making the width of the slot big enough this object was not achieved. In the early stage of a run all the bed load was deposited in slot No. 1, but gradually deposition started in slots No. 2 and No. 3 also;

b. With the gradual deposition of bed load in the slots the vertical velocity distribution also changed indicating that part of the sediment that dropped in slot No. 1 was picked up and carried in suspension to slot No. 2 where it was dropped;

c. There was no appreciable change in the vertical velocity distribution at the centres of the slots above the bed of the canal with and without the slots, but the velocity distribution changed at a certain depth below the bed level.

These experiments indicated that there is a continuous exchange of material between what is called "bed load" and "suspended load".

(b) Calibration of bed-load samplers

The various types of bed load samplers mentioned above require careful calibration before they can be used to measure actual bed-load samples in natural streams or canals. For this purpose they are generally calibrated in laboratory flumes where the bed load is directly measured in a sump at the end of the flume. This has been done by Shamov[2] and Einstein.[3] Models of the Nesper sampler to scales of 1:10, 1:15 and 1:25 were tested in a laboratory flume by Einstein. Definite conclusions could not be drawn from the resulting data. Further the model laws for such experiments have also not been derived as yet.

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1. Punjab Irrigation Research Institute, India, *Annual Report* 1938.
2. Shamov, E.I., "Brief Information Regarding the Results of Laboratory Tests of Silt Samplers", *Transactions of the Scientific Research Institute of Hydro-mechanics, Russia*, vol. XVI 1935, pp. 218-20, in (Russian). Translation No. 183, Department of Mechanical Engineer, University of California, Berkeley, Calif. (USA).
3. Einstein, H.A., "The Calibration of the Bed-Load Trap used in the Rhine River", Translation No. 99-7, US Waterways Experiment Station, Vicksburg, Miss. (USA).

(*c*) *Selection of the proper type of bed-load sampler*

From the above discussions it will be realized that no reliable and accurate bed load sampler has been developed as yet. Attempts have been made by various workers with more or less indifferent success.

The ideal sampler must be capable of doing two things: firstly, it must "cut out" or sample a certain definite portion of the moving stream of water and solids; and secondly, it must collect all the solids from this sampled portion. Such performance can only be assured by careful consideration of the design of the entrance and of the separating mechanism. The proper design of these two features will vary with the conditions in the stream in which they are to be used.

In the ideal sampler, the entrance must not influence the flow upstream in any way and offers no obstruction against the entrance of particles. Furthermore, it must rest securely in contact with the bed while in operation. Generally, the dimensions of the entrance will be governed by the particle size of the bed. Its smallest dimension should be at least twice the maximum grain size, while its width should not exceed 100 to 200 times the average grain size of the bed. The separating mechanism should, whenever possible, make use of screens. However, when the stream carries such large quantities of organic matter that a screen tends to plug up within the time of one sampling period, separation must be based on the principle of local velocity reduction.

The box or basket type sampler is the only one applicable for mountainous streams with coarse gravel as bed material. It is the smallest type for given entrance dimensions, and therefore, the least cumbersome, but it has the disadvantage of creating a considerable back pressure, which causes the water of the slow moving bottom layers to be deflected around the sampler, and tends to retard the quick flowing upper layers. Therefore, the fine material creeping along the bottom is very intensively deflected, while materials of the same grain size, while moving by saltation, is more readily trapped. For sandy beds the pressure difference type of sampler seems to be the most satisfactory, especially when the entrance section is small and the frame flexible enough to guarantee a snug fit against the irregularities of the bed.

The pan-type samplers seem to be best suited for cases with low rates of movement on a comparatively smooth sand bed, when the whole of the transportation is concentrated in the bottom layer.

The final selection of type most applicable to the particular conditions in a given stream can only be based on a thorough calibration test duplicating all the conditions of the river as closely as possible. The calibration of the sampler is, therefore, almost as important as the measurements themselves.

Sampling of bed material

Bed-material samplers collect samples of the material present on the stream bed. As distinguished from bed-load samplers they are simpler in design and easier to handle. The object of these samplers being only to collect samples of the material that comprises the bed, they can be classified as

(*a*) Drag bucket type

(*b*) Vertical core drill type

(c) Grab bucket type

All these samplers function more or less satisfactorily if they are designed properly. The object is only to collect samples from the moving bed without unduly disturbing it or boring too deep into the bed material. When bringing the sample out from the stream, it should not lose any appreciable part of the sample, specially the "non-wash load" portion. Very elaborate arrangements are generally not required of these samplers.

Sampling of suspended load

Sampling of suspended sediments in flowing streams seems to have been practised from early days, utilizing nothing more than a bottle to take samples.

A gradual development of the field practice of taking suspended sediment has taken place, from the simplest methods in the early years to the more complex but more accurate technique in recent times. However, in this field the more primitive methods have persisted along with the new ones to a larger extent than in most scientific lines, and cases of the earliest practices are still found in use at the present time.

The concentration of suspended sediment not only changes from point to point in a cross-section, but also fluctuates momentarily at any fixed point. Depending upon the type of concentration required to be obtained, i.e., whether instantaneous or average concentration at a point or the average concentration in a vertical, a suitable sampler has to be selected according to such requirements. For practical reasons, it is always desirable to make only sufficient measurements which may represent the true mean concentration at a cross-section. It is therefore essential to select a suitable site for observation.

(*a*) *Suspended load sampler*

Though sampling of suspended sediment had been practised from very old days, the requirements of an ideal sampler have been changing with the progress of our knowledge of sediment movement in natural streams. The first and foremost criterion of a good sampler is that it should be able to collect a representative sample from the flowing stream at any desired point. The presence of the sampler and the process of collecting the sample should not disturb the flow of the sediment in the medium from which the sample is drawn. Further the sample of sediment-water mixture should not be allowed to be contaminated by the supply of sediment or water from any part of the stream while the sampler is withdrawn from the medium. The volume of the sediment collected should be sufficient for the purpose of analysis. If this is not so due to paucity of sediment charge in the stream in certain parts of the year, either the size of the sampler should be increased or preferably samples should be collected from the same spot repeatedly so that the bulk of the sediment collected is appreciable. The sampler should be portable and its mechanism should not be very complicated so that if required it can be set right in the field. The sampling arrangement should be such that it should be possible to collect a complete set of samples in the minimum time. These are some of the fundamental requirements of a good sampler. A number of these have been designed and constructed in different countries and operated successfully.

A comprehensive study of 65 existing samplers, developed all over the world, was made by an inter-department committee in the United States. According to report No. 8 entitled *Measurement of the Sediment Discharge of Streams*, published by this committee, these samplers can be classified into the following six general types:

(*i*) Vertical pipe
(*ii*) Instanteneous vertical
(*iii*) Instantaneous horizontal
(*iv*) Bottle
(*v*) Integrating
(*vi*) Pumping

A sampler with a vertical cylinder or pipe for the sample container, not designed to obtain an instantaneous sample, is classified as a vertical pipe sampler. As the sampler is lowered to the desired depth, the water-sediment mixture flows upward through the sample container, and when the lowering is stopped, valves at either end of the pipe close of their own weight and the sample is trapped. While the simplicity of design has been the reason for extensive use of this type in the past, many adverse sampling characteristics have practically eliminated it from current use.

Besides the obvious difference in the sampling period of an instantaneous sampler and an ordinary vertical pipe sampler, the instantaneous sampler obtains a speciment from a smaller part of the vertical after the sampler is lowered to the sampling point. A messenger weight dropped down the suspension line releases the sampler cylinder which falls or is forced down by a spring to a flat plate. Instantaneous vertical samplers are designed to minimize the disturbance of flow in the sampling zone.

An instantaneous horizontal trap sampler consists of a horizontal cylinder equipped with end valves which can be closed suddently to trap a sample at any desired time and depth. Water is allowed to pass through the horizontal cylinder as the sampler is lowered to the desired depth. The principal advantages of this type of sampler are the relative simplicity of design and operation, the ability to sample close to the stream bed, and the wide range of adaptability to shallow and deep stream of all velocities.

The most readily improvised sampling device for sediment investigations consists of an ordinary milk bottle, fruit jar, or other standard container with the necessary provisions for lowering to the sampling point and opening the bottle at the desired depth. The bottle type sampler is provided with an entrance varying in size from about ½ in diameter up to the full opening of the container. Air within the bottle, displaced by the incoming sample, escapes through the intake opening and thus produces a bubbling action at the entrance. Because of the time required to fill the sampler and because of the air bubbles which escape through the intake, bottle samplers are often referred to as slow-filling or bubbling samplers. This is the type in common use in countries of the ECAFE region.

The container of an integrating sampler is filled slowly and continuously over a period of time ranging from about 10 to 60 seconds. An important feature of most of the integrating samplers, distinguishing them from the ordinary bottle samplers, is that the intake and the air exhaust tubes are separated. Thus the escaping air causes no disturbance to the inflow as the sampler is filled. The intake tube or orifice is pointed into the current so as to eliminate change in direction of the flow.

In the operation of a pumping sampler, the water-sediment sample is sucked in through a pipe or hose, the intake of which is placed at the desired sampling point. By directing the intake of the suction hose into the current and regulating the intake velocity, the operator can obtain an undisturbed sample representative of the sediment concentration at the sampling point.

The outstanding characteristics of the different types of samplers are summarized in table 16.

Table 16

Characteristics of Suspended Sediment Samplers

Type	Disturbance of flow characteristics	Intermixing of sample with water	Sampling action	Field handling of sample	Adaptability to various field conditions
Vertical pipe	Excessive	Generally excessive	Instantaneous	Necessary transfer another container	Offers considerable resistance to current. Not satisfactory close stream bed
Instantaneous vertical	Questionable	None	Instantaneous	Necessary transfer another container	Not satisfactory streamlined or adapted for use near stream bed
Instantaneous horizontal	Tendencies minimized, Effect not evaluated	Slight possibility	Instantaneous	Necessary transfer another container	Allows sampling very close to stream bed. Adaptable to any stream or depth
Bottle	Excessive effect not evaluated	Excessive if not opened and closed at site	Bubbling or slow-filling after initial rush	Container with sample removable	Not capable of sampling close to bed of stream
Time-integrating	Tendencies minimized but not evaluated	Some extent if not opened and closed at site	Smooth filling, initial inrush	Container with sample removable	May be limited by depth of stream
Pumping	Tendencies minimized with proper control of intake tube and velocities	None	Time-integrated	Container with sample removable	Present design not portable. Somewhat limited in use due to resistance to current. For heavey sediment loss in pipe line may limit use.

Suspended sediment samplers can also be classified according to their mode of operation, into instantaneous and integrating samplers. Many samplers do not actually qualify for either classification because their design, sampling action, or method of operation eliminate them from the categories of true instantaneous or integrating samplers. As the name implies, the instantaneous sampler is designed to trap a speciment of the water-sediment mixture passing the sampling point at a desired instant. On the other hand, the integrating sampler take the sample over an extended period so as to obtain either a specimen at a point in which the momentary fluctuation in sediment concentration are averaged or a specimen in a vertical in which the concentration at different depths are averaged. Integrated samples are obtained with either a point-integrating sampler or a depth-integrated sampler.

In order to obtain a representative sample, it is essential that the flow limes at the intake of the sampler must be undisturbed during the sampling period and that the velocity of inflow into the sampler should be the same as the current velocity at the point of intake.

In this respect, the point integrating sampler US P-46 and the depth integrating samplers US D-43 and D-47 developed by the aforementioned sub-committee fully meet the requirements and can be adopted for the measurement of suspended sediment.

(*b*) *Location of vertical sampling stations in a cross-section*

Methods used in locating vertical sampling stations have varied with different investigations. They can be classified as follows:

(*i*) Single vertical at midstream
(*ii*) Single vertical at the deepest point
(*iii*) Vertical at $\frac{1}{4}$, $\frac{1}{2}$ and $\frac{3}{4}$ width
(*iv*) Vertical at $\frac{1}{6}$, $\frac{1}{2}$ and 5/6 width
(*v*) Four or more verticals equally spaced across the stream
(*vi*) Vertical at middle of section of equal discharge

The selection of these different positions of the vertical and the number has been dictated more by urgency of the work as well as by the accuracy required. It is obvious that the more the verticals chosen the more accurate will be the final assessment of sediment charge, but time being an important factor in all these observations, some selection will have to be made to get the best results in the shortest time. The method of selecting vertical sampling stations as developed by E. W. Lane which is based on the principles of the Luby method for selecting sampling points in a vertical, appears to be rational. When reliable data concerning the distributions of the discharge at the sampling sections, for all stages of the stream, are available, the verticals are so chosen that each segment of area bounded by lines midway between two adjacent verticals will represent an equal portion of discharge.

(c) Selection of sampling points in a vertical

The variation of sediment concentration from the bottom to the surface of a stream is, in general, more considerable than that across the section. In most cases the concentration is found to be more in the deepest vertical and in the middle of the stream if the section is not too asymetrical. Some of the extreme variations of sediment concentration are very often due to momentary fluctuations in turbulence of the stream and very difficult to explain. In steady flows and also at stretches away from any sudden change in regime of the channel, such abrupt fluctuations are very rare. As in the case of selection of verticals for taking suspended samples so also in selecting points of sampling in the verticals, the more the number of points in a vertical at which samples are taken the better is the chance of getting a representative sample. This, however, is limited by time and expense factors. Variations therefore in the vertical distribution of sediment have been taken into account in certain investigations in either of the following ways:

(*i*) By taking samples at enough points to establish the vertical variation with the required degree of accuracy;

(*ii*) By applying correction factors, based on previous observations, to samples taken at definite locations;

(*iii*) By taking samples with the depth-integration sampler which intergrates the concentration throughout the depth.

Velocity variations in the vertical have to be given proper weight either by taking samples representing equal volumes of discharge, or by taking samplers representing known volumes of discharge.

Procedures suggested under (*i*) and (*ii*) above appear to be rational. But to arrive at any reasonable criterion for the determination of the average point, a number of preliminary observations are generally required. It is also sometimes necessary to repeat these preliminary investigations during the course of the experiments if any fundamental change in the region of the channel is suspected.

As regards (*iii*), the depth-integration method of sampling suspended sediment in stream presupposes that the sampler fills at a rate proportional to the velocity of the approaching flow and that in traversing the depth of a stream at a uniform speed, the sampler will receive at every point in the vertical a small instantaneous specimen of the water-sediment mixture the volume of which is proportional to the instantaneous velocity.

In depth-integrating sampling a slow filling type of sampler must be used. In using this method it has frequently been the practice to lower a closed sampler to the bottom, then open it and raise it at such a rate that the sampler would reach the surface before the container was entirely filled, thus assuming that water from every point in the vertical was taken in. When a sampler with a fixed-volume container is operated in this way a portion of its capacity is filled in a moment after the valve is opened. This initial filling, which is induced by the difference in pressure inside and outside the container is therefore proportional to the depth of submergence. It is evident that during this period the filling rate is not a function of the velocity and, hence, the sample as a whole is not representative of the average water sediment mixtures in the vertical. To avoid this difficulty the sampler should be open during the descent as well as the ascent and should be lowered and raised at a uniform rate such that the round trip is made before the container is entirely filled. This procedure does not endure a perfectly integrated sample, but, for practical purposes, it is probably satisfactory, if the stream is not too deep to permit the round trip to be made before the container is filled.

J. P. Luby[1] developed a rational method for selecting points in the vertical which has been used recently by some of the other US Engineer Districts. In this method the area under the vertical velocity curve is divided into equal areas. If samples are taken at the centres of these areas they will be representative of equal portions of the discharge. The samples, if of equal volume, can be combined and the composite sample will be representative of the mean sediment concentration in the vertical. The mean particle size composition will also be represented in the composite sample, provided enough points are chosen. The number of points to sample will largely be determined by the curvature of the sediment distribution curve, the slope of the vertical velocity curve and the depth of the stream.

Table 17 gives a comparative summary of the various methods in use for observing suspended sediment in a vertical.

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1. "Field Practice and Equipment used in Sampling Suspended Sediment", Report No. 1 in *A Study of Methods . . . op. cit.*

Table 17

Method of selecting sampling points in a vertical

Method and description	Discussion	Reliability and accuracy for determining		Practical considerations	Number of samples and analyses per vertical
		Concentration only[a]	Concentration and particle size		
Single-point A single sample secured at the surface.	Arbitrary method unless coefficients have been determined from previous, more complete sampling, then it is somewhat empirical.	Not reliable or necessarily accurate even when a coefficient has been determined.	Not at all reliable or accurate.	Simplest of all present methods, rapid and easy to use. Readily adapted for use by unskilled observers. Requires previous more exact sampling for justification.	One sample and one laboratory analysis.
Single-point A single sample secured at any point in the vertical other than the surface.	Arbitrary method unless coefficients have been determined from previous, more complete sampling, then it is somewhat empirical. A common point to sample has been 0.6 depth.	Generally not reliable or accurate even when a coefficient has been determined, but more so than a single surface sample. Thoroughness of preliminary investigations will determine, somewhat, the reliability and accuracy.	Not reliable or accurate.	Simple, rapid, and easy to use, but fractional depth measurements make it less adaptable to use by unskilled observers than single surface method. Requires previous, more exact sampling for justification.	One sample and one laboratory analysis.
Two-point Two points selected arbitrarily for convenience and adaptability to the skill of the observer.	Arbitrary method with no rational justification.	Generally not reliable or accurate for all conditions of a given stream.	Generally not reliable or accurate.	Fairly simple, rapid, and easy to use. Can be used by dependable observers even though in-experienced.	Two samples may be combined if of equal volume for a single analysis.
Three-point Arbitrary selection of points at surface, mid-depth, and bottom with equal weights.	Points located arbitrarily.	Not necessarily reliable or accurate for all stream conditions.	Not necessarily reliable or accurate.	Sampling at surface, mid-depth, and bottom is the most simple and easiest to use of all methods requiring more than two samples. Can be used by dependable observers even though in-experienced.	Three samples may be combined if of equal volume for a single analysis.
Three-point Arbitrary selection of points at surface, mid-depth, and bottom with weights of 1, 2, and 1 applied, respectively.	Basis of method is assumption that the averages of surface and mid-depth samples represents upper-half of discharge and average of mid-depth and bottom represents lower half.	Not necessarily reliable or accurate for all stream conditions.	Not necessarily reliable or accurate, but more so than three points, surface, mid-depth, and bottom with equal weights.	Sampling at surface, mid-depth, and bottom is the simplest and the easiest to use of all methods requiring more than two samples. Can be used by dependable observers even though in-experienced.	Three samples. If of equal volume, surface and bottom samples may be combined for single anlysis.

a. For methods where coefficients are used, comments apply only to individual observations or short period investigations, as over long periods, totals may have a fair degree of accuracy.

Table 17 (Continued)

Method and description	Discussion	Reliability and accuracy for determining		Practical considerations	Number of samples and analyses per vertical
		Concentration only[a]	Concentration and particle size		
Precise A relatively large number of point samples at known locations in each vertical simultaneous with velocity measurement.	Rational method for use primarily in special investigations. Number of sampling points depends upon depth of stream, the velocity and sediment distribution, and the degree of accuracy desired.	Reliable and accurate. Accuracy depends upon the curvature of the velocity and sediment distribution curves and number of samples. The most accurate method in use at present.	Reliable and accurate. Accuracy depends upon curvature of particle distribution curves, and number of samples. The most accurate method in use at present.	Not adapted to routine sampling because of the excessive work required. Its use is limited to research or preliminary investigations. Laboratory work excessive as all samples must be analyzed separately.	Minimum of four or five samples all to be analyzed separately.
Straub Sampling at 0.2 and 0.8 depth, applying coefficient obtained by mathematical derivation for both linear and curvilinear sediment distribution. For linear distribution values weighted 5/8 and 3/8 for 0.2 and 0.8 depth, respectively.	Rational method, best adapted for use where the vertical sediment distribution curve approximates a straight line, and the velocity distribution is fairly constant.	Accuracy and reliability depends almost entirely upon the agreement of the actual to the assumed sediment and velocity. In most cases quite reliable.	Theoretically not sound if sediment distribution is curvilinear, but, practically, one of the most reliable methods.	Field work relatively simple for skilled observer but adaptable also to dependable observers, even though in-experienced.	Two samples and two analyses.
Luby Sampling points selected at the middle of increments of depth representing equal portions of stream discharge.	Rational method if a sufficient number of samples are collected. The samples, if of equal volume, can be combined and composite representative of the mean concentration and composition in the vertical. Number of points with respect to depth depends primarily upon curvature of sediment distribution curve; to a lesser extent, generally, upon curvature of vertical velocity curve.	Reliable and accurate if a sufficient number of samples are collected. One of the most reliable and accurate of the present methods except the precise.	Fairly reliable and accurate if a sufficient number of samples are collected. One of the most reliable and accurate of the present methods except the precise. Enough samples should be taken so that one will be close to the stream bed.	Requires either an assumed velocity distribution or previous velocity measurements. Too complicated for use by trained hydrographers. Because of sampling more points a better representation of the actual sediment distribution will probably be obtained than with the Straub method.	Minimum of five samples. May be combined if of equal volume for a single analysis.
Depth-integration Single sample collected from all points in the vertical usually obtained by lowering and raising a slow filling sampler at constant rate These usually consists of ordinary milk bottle types or specially designed slow-filling samplers.	Rational method only if sample is collected proportional to velocity.	Relatively reliable under usual conditions but its accuracy varies as most of the present equipment does not sample proportional to the velocity and many samplers do not approach close enough to the bottom. As used, accuracy depends upon depth of stream and type of sampler.	Relatively reliable under usual conditions but its accuracy varies as most of the present equipment does not sample proportional to the velocity, and many samplers do not approach close enough to the bottom. As used, accuracy depends upon depth of stream and type of sampler.	As commonly used with simple slow-filling samplers this method is simple, rapid, easy to use, and well adapted to dependable observers, even though in-experienced. No previous measurements necessary.	One sample and one analysis.

(*d*) *Frequency of sampling*

The frequency of sampling depends, among other factors, on

(*i*) The purpose of the investigation

(*ii*) The cost of the various procedures

(*iii*) Variations in the sediment content of the stream

The purpose for which sediment samples are required is one of the main considerations that generally determines the duration and frequency for the collection of the samples. In a project like multiple-purpose river basin development for flood control, irrigation, power generation and river conservancy, it is most essential for the designing engineer to know the sediment charges of the stream for a number of years taken systematically throughout the year. For such a purpose it may also be necessary to collect relevant information about sediment characteristics of the different streams feeding the main river on which the dam is proposed to be built. For such a project the engineers cannot have too much sediment data—the more the information they have the better; but the data must be reliable and correctly collected. Otherwise the data may be worse than useless. They may be even harmful.

The cost of collection of the sediment data will also have to be taken into consideration, but it should not be the governing factor.

While drawing up a programme of sediment observation for any project due regard should be paid to the variation of the sediment content in the stream flow due to various factors. In countries where the flow in rivers is governed by monsoonic conditions, sediment content is very heavy only during the rainy season while during the dry season when the flow is entirely due to regeneration from seepage flow the sediment charge may be nil, whereas if the river is fed by snow-melting, the sediment will consist mostly of suspended and soluble material. If it is not desired to collect information about PH or chemical content of the river water, sampling during the dry season may be entirely discontinued or limited to only one sample a day from one vertical in one cross section only.

ANALYSIS OF SEDIMENT

The samples of sediment collected by the various methods elaborated before have to be analyzed for grain size distribution so that the knowledge thus gained may be utilized for the control of sediment movement. It will be realized that of all the characteristics of sediment charges, the following three will be of immediate concern for our problem:

1. Total quantity or intensity of concentration of suspended sediment
2. Grain size distribution
3. Compactness or volume weight relationship.

The quantity of sediment in a sample or the concentration of suspension can be determined by separating the sediment from the mixture. The volume weight relationship is required for deposited material only. The most difficult is the determination of grain size distribution.

Of the three types of samples, suspended sample, bed sample and deposited sample, the suspended samples present the greatest difficulty in analysis due to the comparatively low concentrations that generally exist. In most of the samples the total sediment concentration, having a large fraction "wash load" varies from 2 g/l to 0.02 g/l (i.e., 0.2 per cent or 2,000 p.p.m. to 0.002 per cent or 20 p.p.m.). The difficulty of accurately separating these small amounts into particle size without excessively laborious procedures is obvious.

It has long been recognized that the fundamental property governing the action of a suspended sediment particle is not its size, but rather the rate at which the particle will settle in still water, which rate is closely related to the size. A recent study[1] had shown that the movement of material in rolling or sliding along the bed also is related more closely to its settling rate than to its size. Therefore, from the viewpoint of hydraulic engineers, settling rate appears to be the most important property of sediment carried by flowing water.

It is however much easier to form a reasonably accurate mental picture of a particle of a given size than of a certain settling rate. Although in the future the settling rate will no doubt be more widely used as the basis of sedimentary particle classification, for the present particle size is being used in most cases as the classification basis, as the settling rate can be determined for the known particle size by means of a fairly definite relation between particle size and settling rate.

A study[2] of all available data on size analysis methods showed that, although no single method was found satisfactory over the wide range of samples of suspended sediments found in streams, the following methods have been found suitable in certain conditions and have been extensively used in sediment studies: sieves, decantation, pipette, hydrometer, siltometer and microscope.

1. Krumbein, W.C.: "Settling Velocity and Flume Behaviour of Non-spherical Particles", *Transactions of the American Geophysical Union*, 1942.
2. "Methods of Analysing Sediment Samples", Report No. 4 in *A study of Methods . . . , op cit.*

Sieve analysis, either wet or dry, is limited to particles of approximately 1/16 mm diameter or larger. Analysis with fine screens is generally not practicable due to the difficulty of constructing accurate screens with smaller mesh. It often happens that the amount of material coarser than 1/16 mm in a suspended sediment sample is so minute as to preclude the possibility of obtaining an accurate sieve analysis. For sediment studies sieve analyses are less satisfactory than the methods of measurement by settling rate, as the relation between sieve size and settling rate is not accurately known.

Although the decantation method is suitable for fine particles and low concentration, the method is laborious and time consuming. The speed with which the coarser particles fall in water make it difficult to analyse material above 1/16 mm diameter by this method.

For material below 1/16 mm the pipette method is considered as the most accurate. The pipette analysis is usually done with a concentration of 2 per cent by weight; but pipette analysis can also be used for higher concentrations.

The hydrometer provides a simple method of size analysis and has found wide use in connection with the construction of earth dams, levees, highways and in other applications of soil mechanics. Within the range where it is satisfactory, it is perhaps the simplest and most rapid method, although it is probably less accurate than the pipette method. As in the case of the decantation and pipette methods, it is impracticable to analyse sediment coarser than about 1/16 mm with the hydrometer.

Size analysis by settling rate for coarser particles has been widely used in India. The "optical lever siltometer" was developed by Vaidhianathan[1] in 1933. A mercury manometer located near the bottom of a long sedimentation tube indicates changes in pressure with time after a concentrated sample has been introduced at the top of the tube. The movement of the top of the mercury column is generally enlarged by reflecting a beam of light upon a photographic paper on a revolving drum. The apparatus was designed for a size range of 0.075 mm to 0.60 mm diameter particles.

In 1934, Puri[2] designed a siltometer for analysing sediment samples with particle diameter between 0.06 mm to 0.6 mm in which the sample is introduced at the top of a long tube of water and fractions withdrawn from the bottom.

In all these methods of sediment size range analysis, water has been the medium used for the purpose. Suggestion for using wind tunnel as the classifier for sand and silt has been made by Rouse.[3] The technique and results described by Uppal[4] appear to be satisfactory, though the apparatus will require careful calibration frequently.

In all these apparatus the method of deriving particle sizes from the rate of fall of the particles had been mainly based on Stoke's law for single particles as modified by Zahn or Goldstein. The effect of temperature on this rate of fall had been taken account of by temperature correction, while the effect of the wall of the tube had been eliminated by making the diametre big enough. It has been recognized that the concentration of the suspension will have some effect but it is only recently that this has been quantitatively assessed. Mcknown[5] has shown that concentration of particles in suspension affects the terminal fall velocity to a significant degree.. A concentration of only 1 per cent was found to reduce the terminal fall velocity by about 20 per cent. For values of Reynold's number greater than about 0.1, the effect of concentration was found to decrease slightly. In view of this finding it may be necessary to modify the technique adopted in siltometres developed by Vaidhianathan, Puri and others.

1 Vaidhianathan, V.I.: "An Optical Lever Siltometer", Paper No. 167, *Proceedings of the Punjab Engineering Congress*, 1933, vol. XXI.

2. Puri, A.N.: "A Siltometer for Studying Size Distribution of Silts and Sands", Punjab Irrigation Research Institute, *Research Publication*, vol. 2, No. 7, 1934.

3. Otto, G.H. and Rouse, Hunter, "Wind-Tunnel Classifier for Sand and Silt", *Civil Engineering*, vol. 9, July 1939. pp. 414-415.

4. Uppal, H.L. and Singh, Gajinder, "An Improved Sand Classifier", *Transaction of the American Geophysical Union*, vol. 32, No. 6, Dec. 1951.

5. Mcknown, John S. and Lin, Pin-Nam, "*Sediment Concentration and Fall Velocity*", Iowa Institute of Hydraulic Research, State University of Iowa, Iowa City (USA).

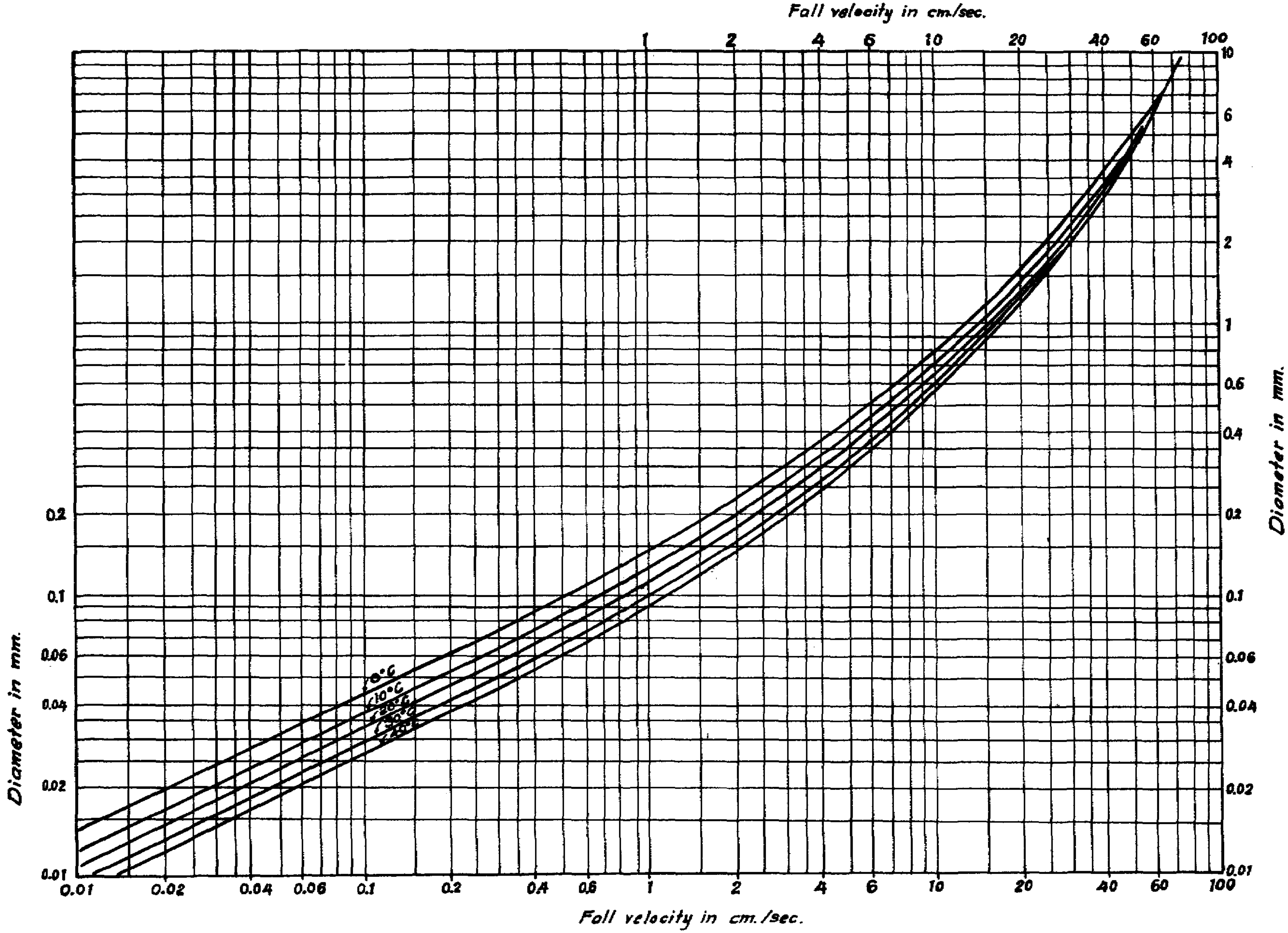

FIG. 1 THE TERMINAL FALL VELOCITY OF QUARTZ PARTICLES IN RELATION TO THE DIAMETER

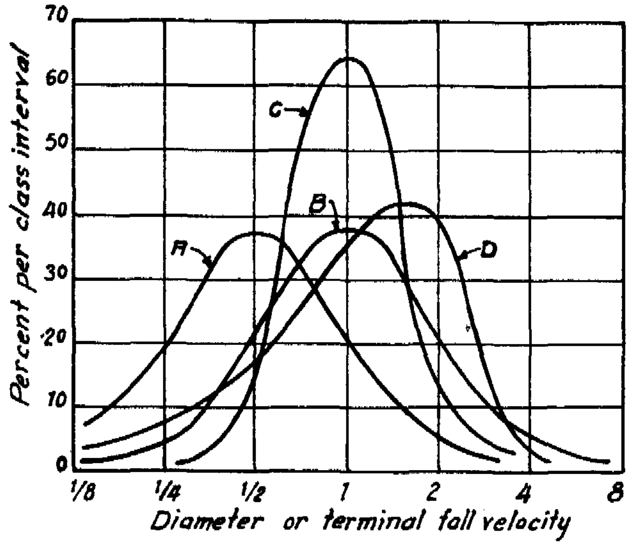

FIG. 2 DISTRIBUTION CURVES OF SEDIMENT

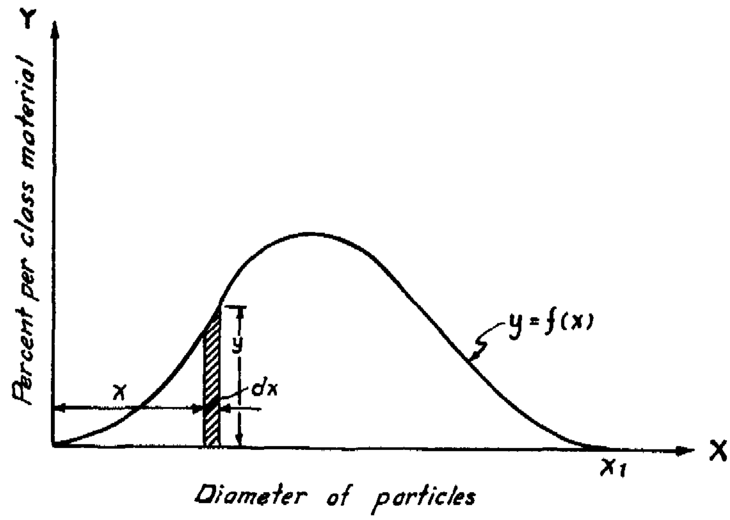

FIG. 3

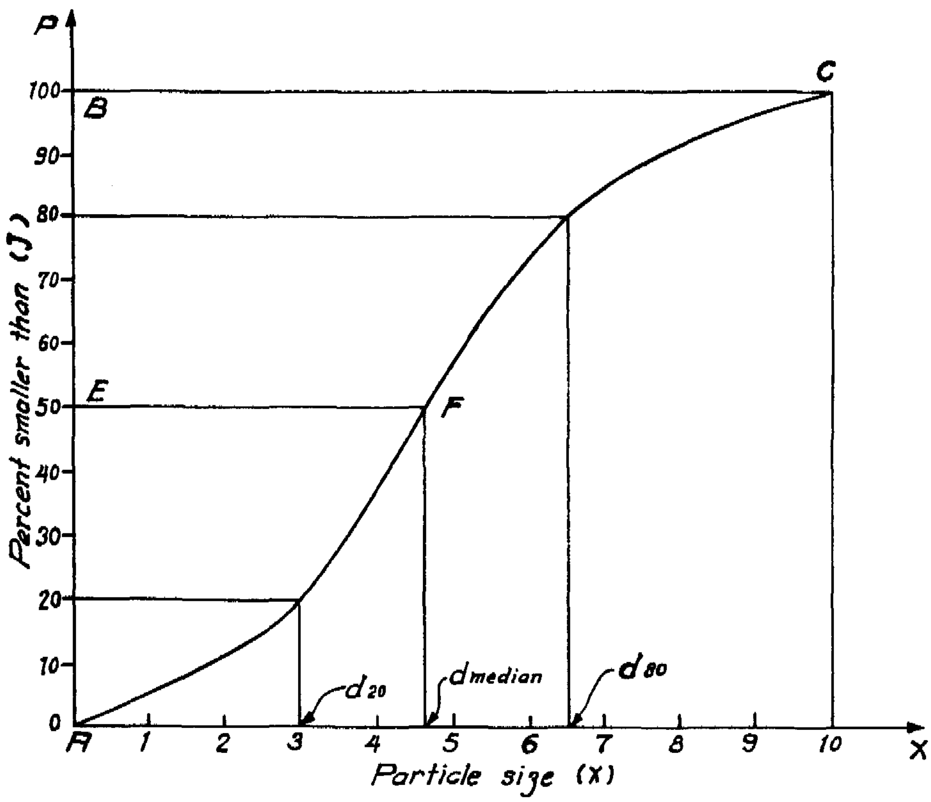

FIG. 4 CUMULATIVE CURVE OF SEDIMENT

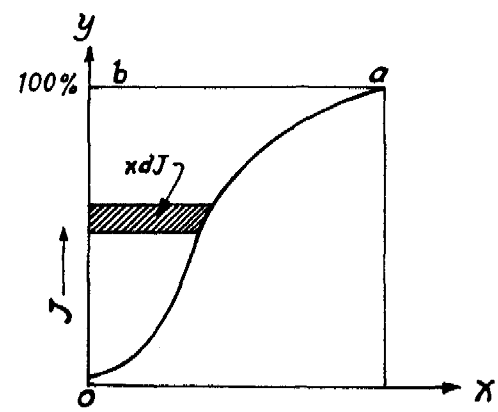

FIG. 5

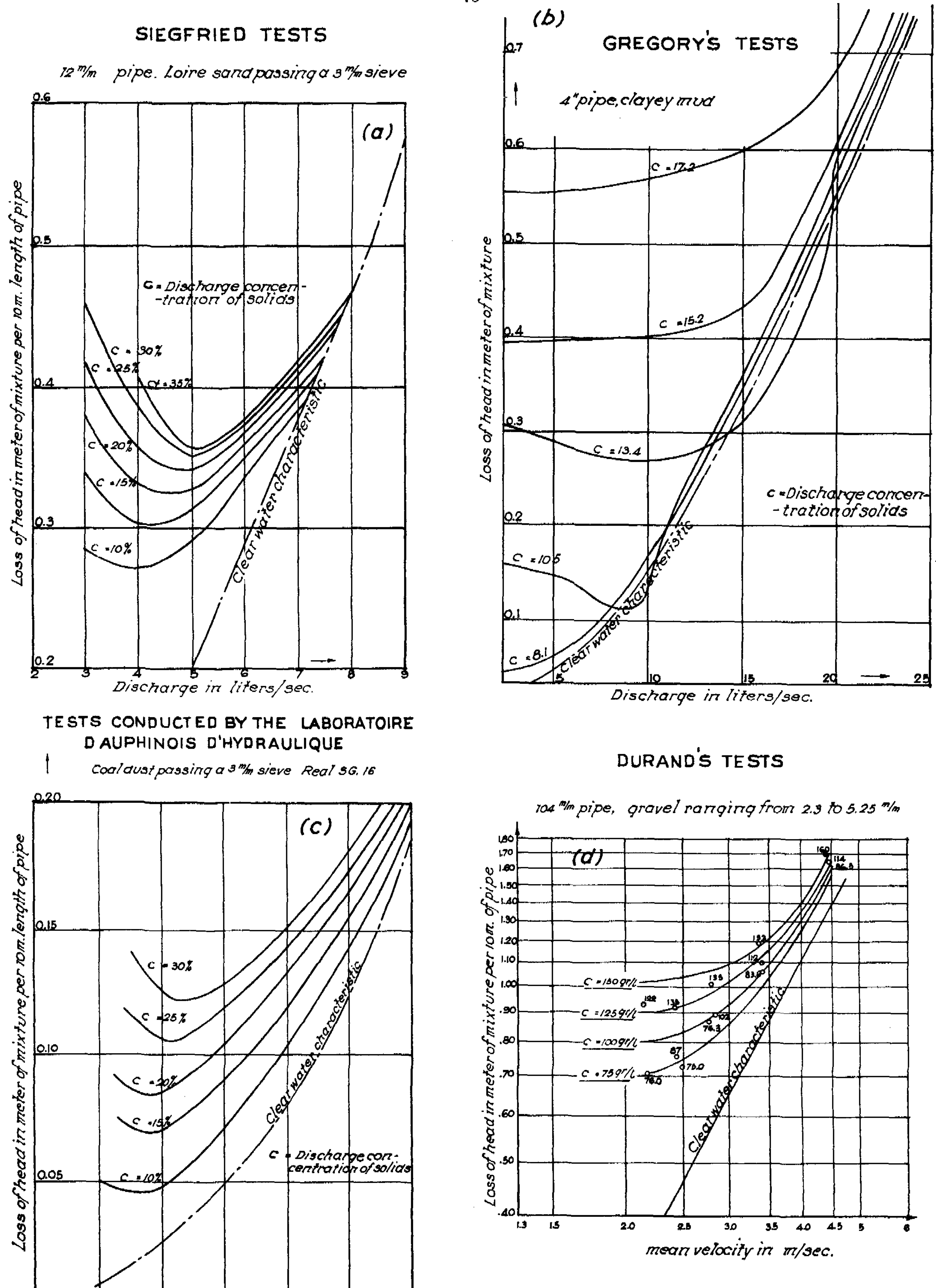
SIEGFRIED TESTS
12 m/m pipe. Loire sand passing a 3 m/m sieve
(a)
C = Discharge concentration of solids
C = 30%
C = 25%
C = 35%
C = 20%
C = 15%
C = 10%
Clear water characteristic
Loss of head in meter of mixture per 10 m. length of pipe
Discharge in liters/sec.
(b)
GREGORY'S TESTS
4" pipe, clayey mud
C = 17.2
C = 15.2
C = 13.4
C = Discharge concentration of solids
C = 10.6
C = 8.1
Clear water characteristic
Loss of head in meter of mixture
Discharge in liters/sec.
TESTS CONDUCTED BY THE LABORATOIRE DAUPHINOIS D'HYDRAULIQUE
Coal dust passing a 3 m/m sieve Real SG. 1.6
(c)
C = 30%
C = 25%
C = 20%
C = 15%
C = 10%
Clear water characteristic
C = Discharge concentration of solids
Loss of head in meter of mixture per 10 m. length of pipe
Discharge in liters/sec.
DURAND'S TESTS
104 m/m pipe, gravel ranging from 2.3 to 5.25 m/m
(d)
C = 150 gr/l
C = 125 gr/l
C = 100 gr/l
C = 75 gr/l
Clear water characteristic
Loss of head in meter of mixture per 10 m. of pipe
mean velocity in m/sec.

FIG. 6

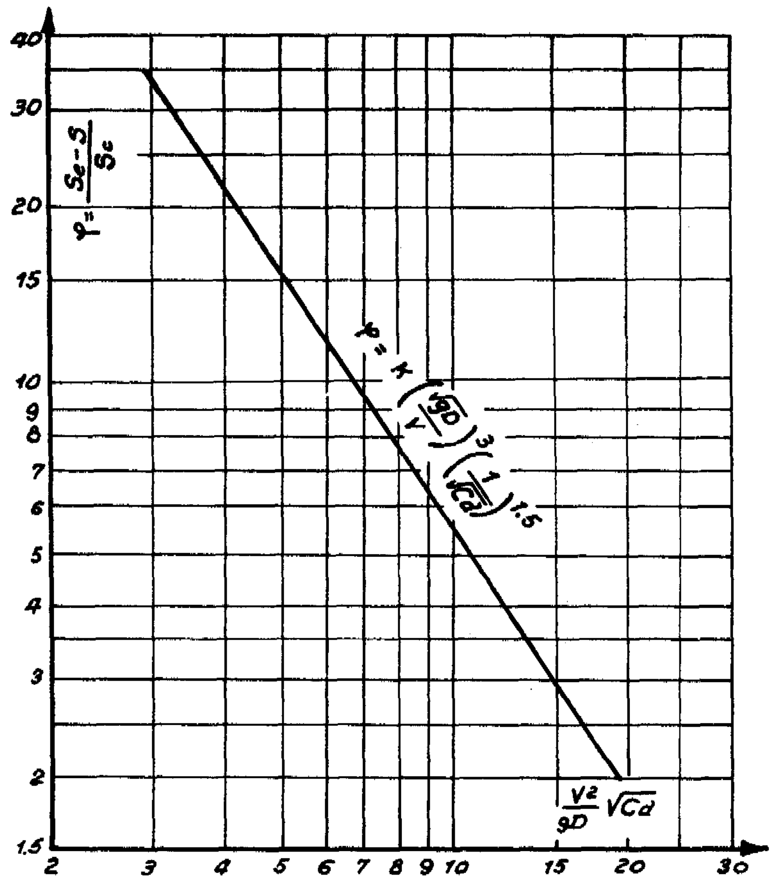

FIG. 7 HEAD LOSSES IN PIPES WITH NON-DEPOSIT FLOW REGIMES (AFTER DURAND)

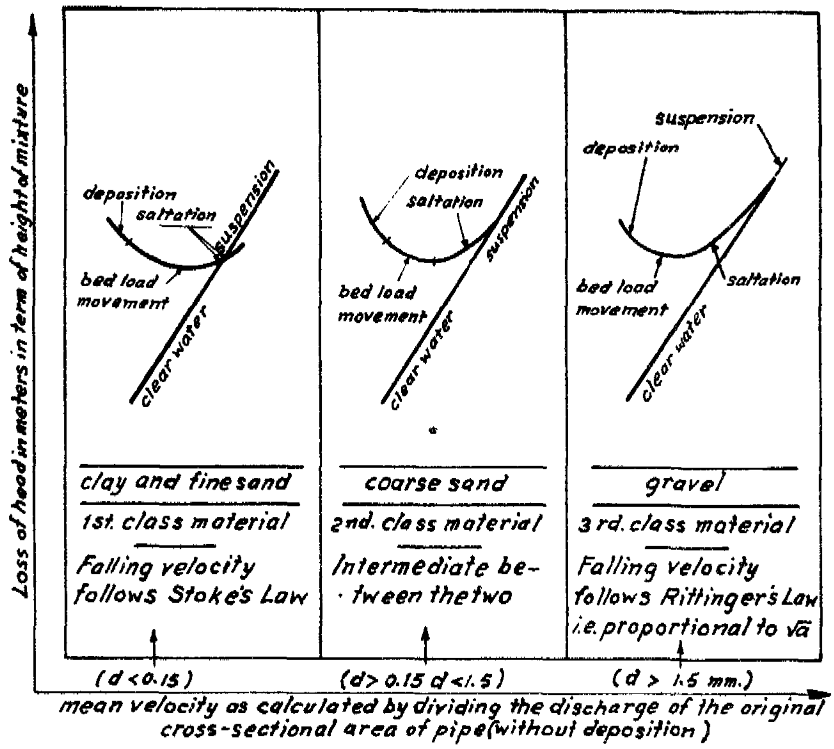

FIG. 8 GENERALIZED DIAGRAM OF SEDIMENT TRANSPORTATION IN PIPES (LOGARITHMIC COORDINATES)

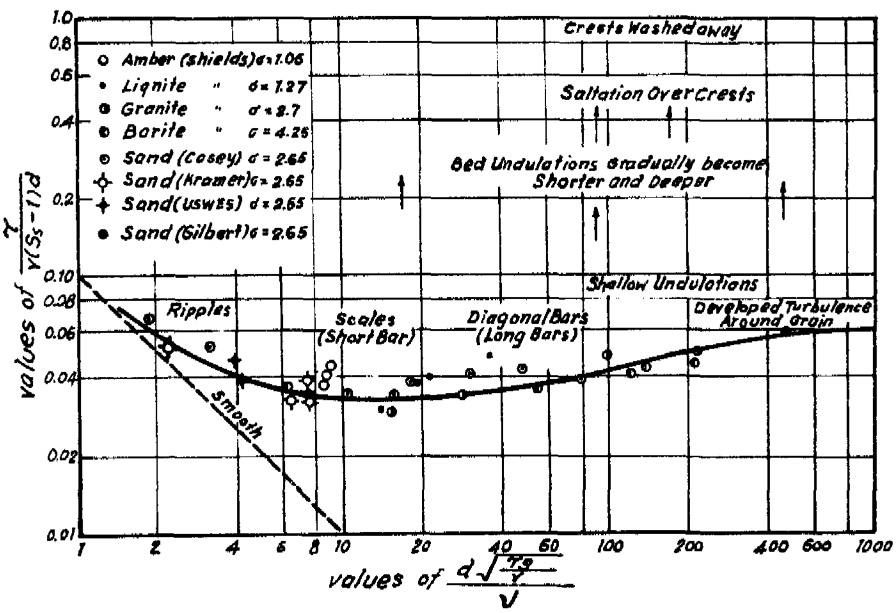

FIG. 9 TRACTIVE FORCE PLOTTED AGAINST REYNOLD'S NUMBER OF SAND GRAIN ACCORDING TO SHIELD'S

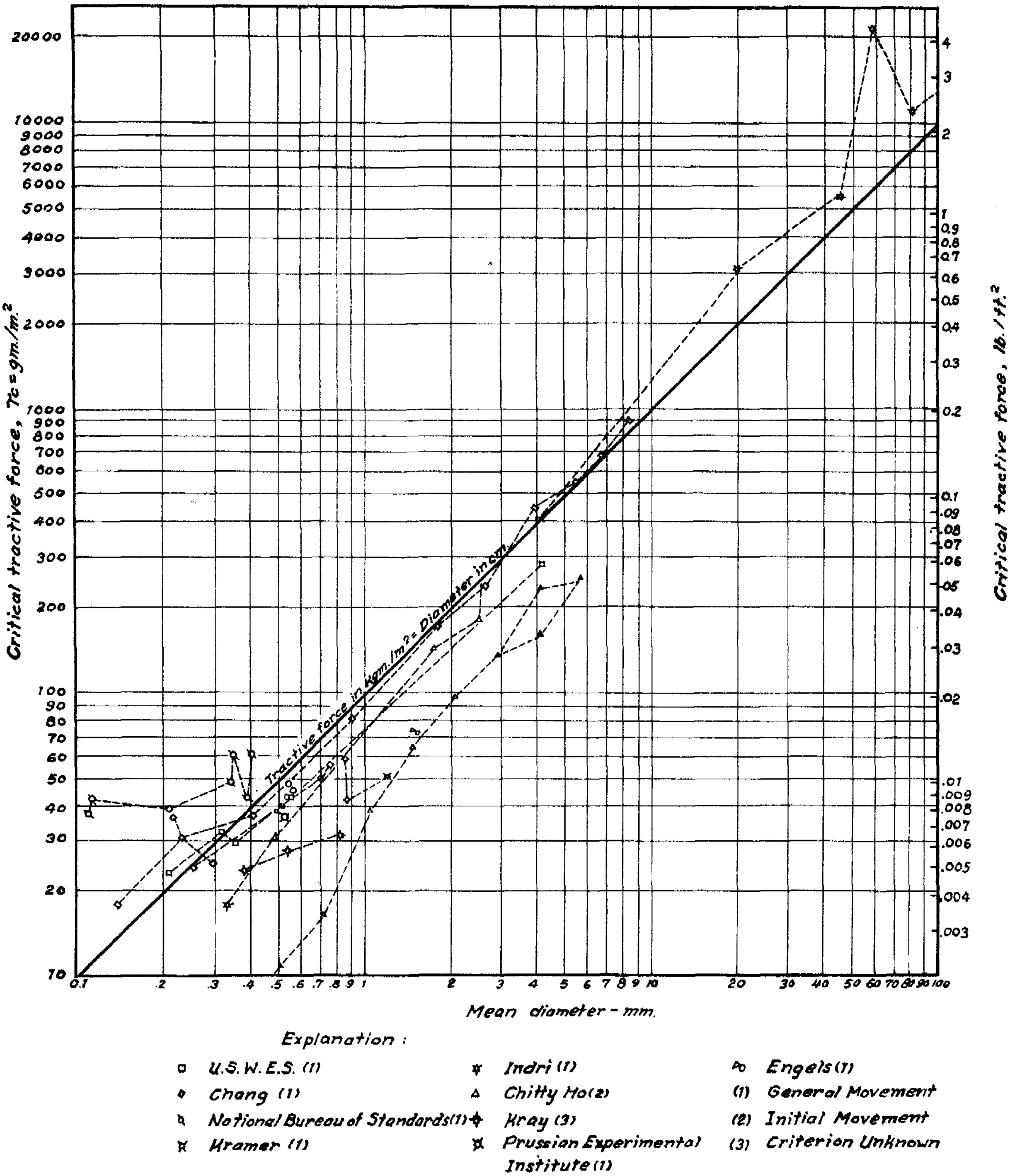

FIG. 10 OBSERVED CRITICAL TRACTIVE FORCE

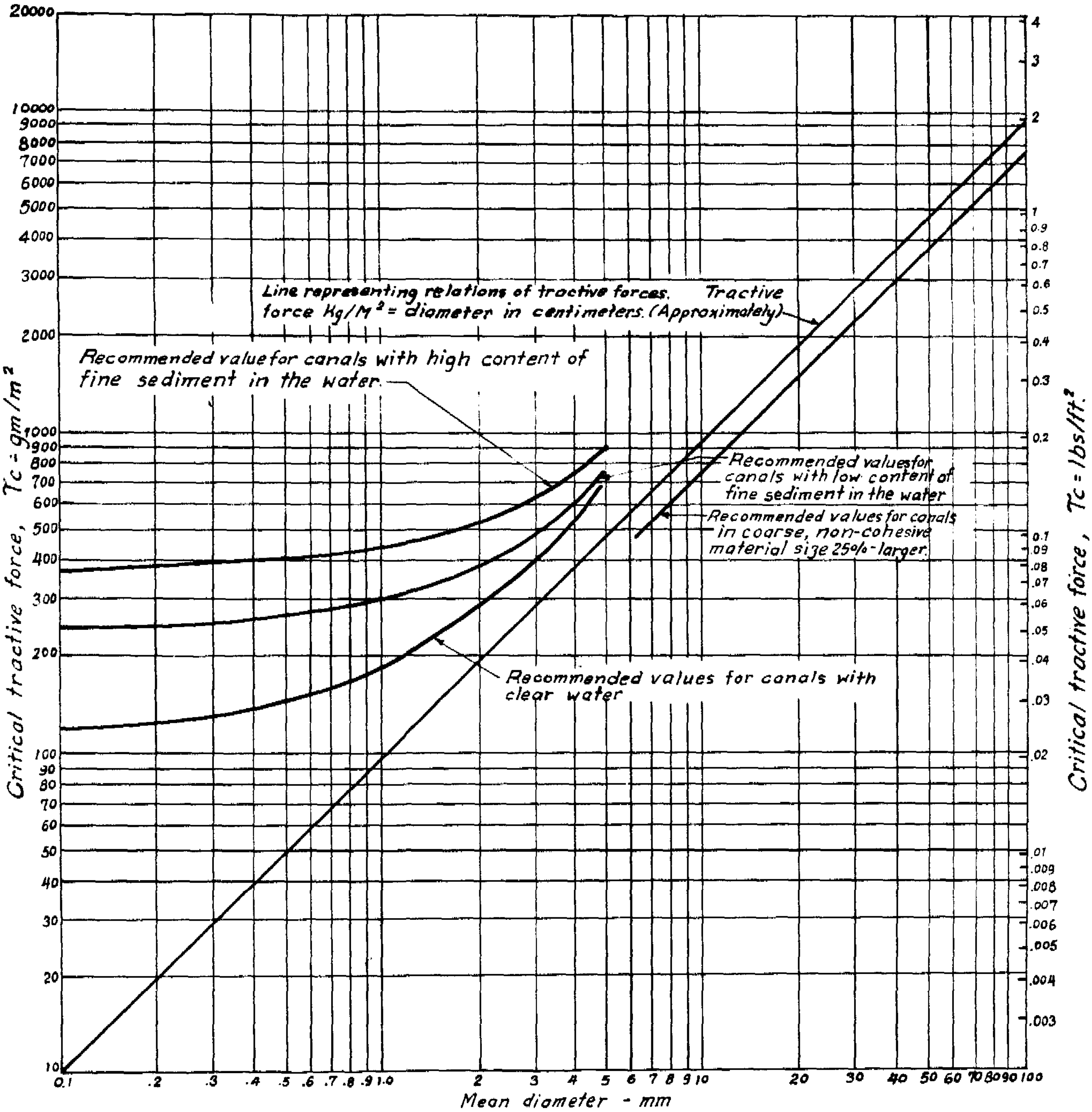

FIG. 11 LIMITING TRACTIVE FORCE RECOMMENDED FOR CANALS

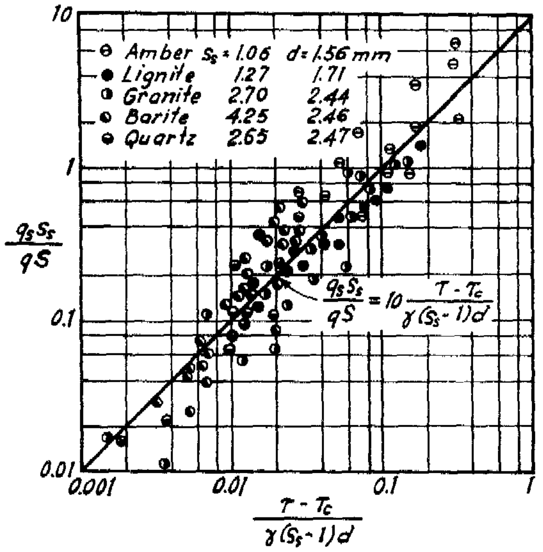

FIG. 12 PLOT OF SHIELD'S BED LOAD FORMULA

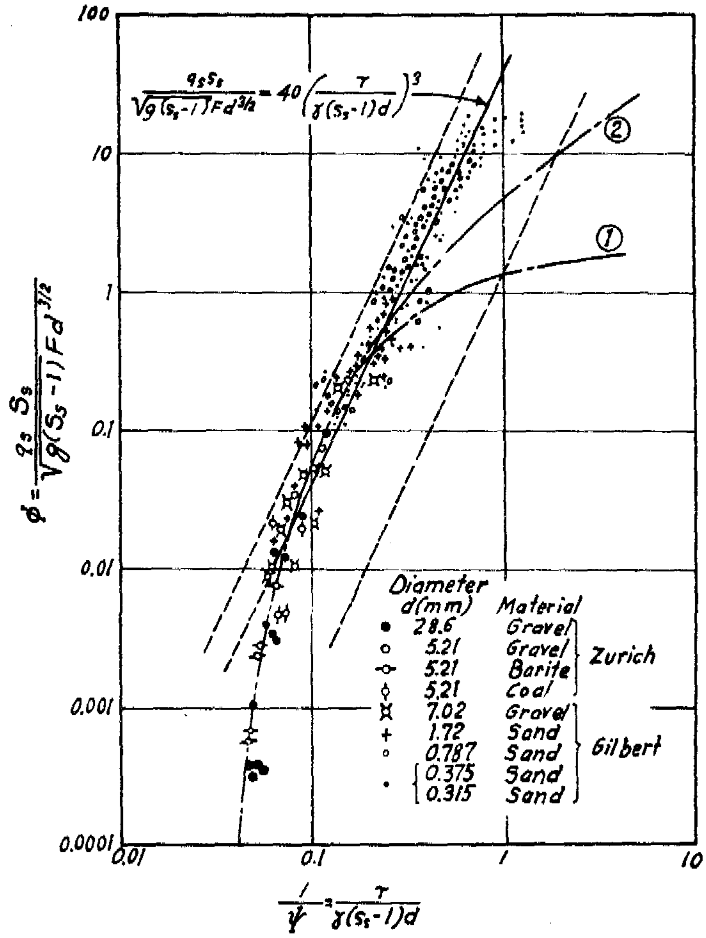

FIG. 15 PLOT OF EINSTEIN'S BED LOAD FUNCTION $\phi - \frac{1}{\psi}$ ACCORDING TO ROUSE

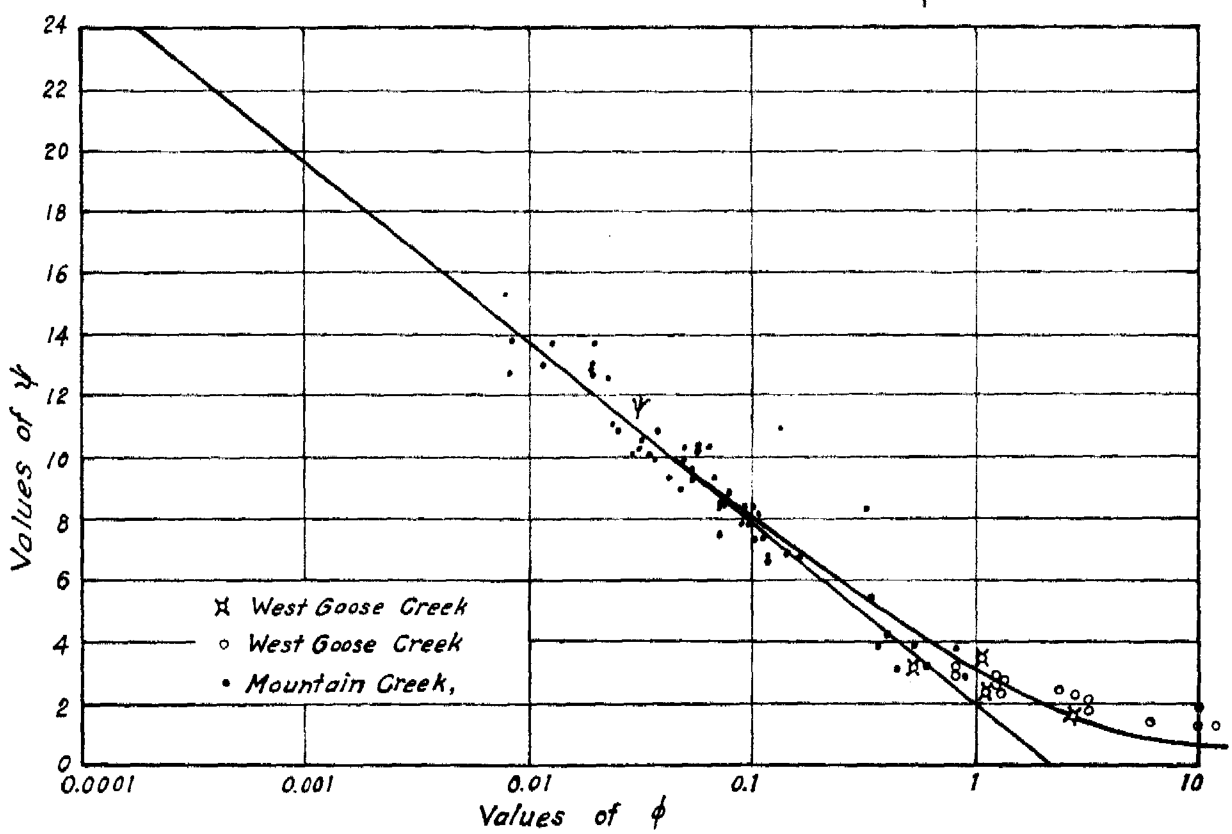

FIG. 14 BED LOAD MEASUREMENTS IN TWO STREAMS PLOTTED ACCORDING TO $\phi - \psi$ FORMULA

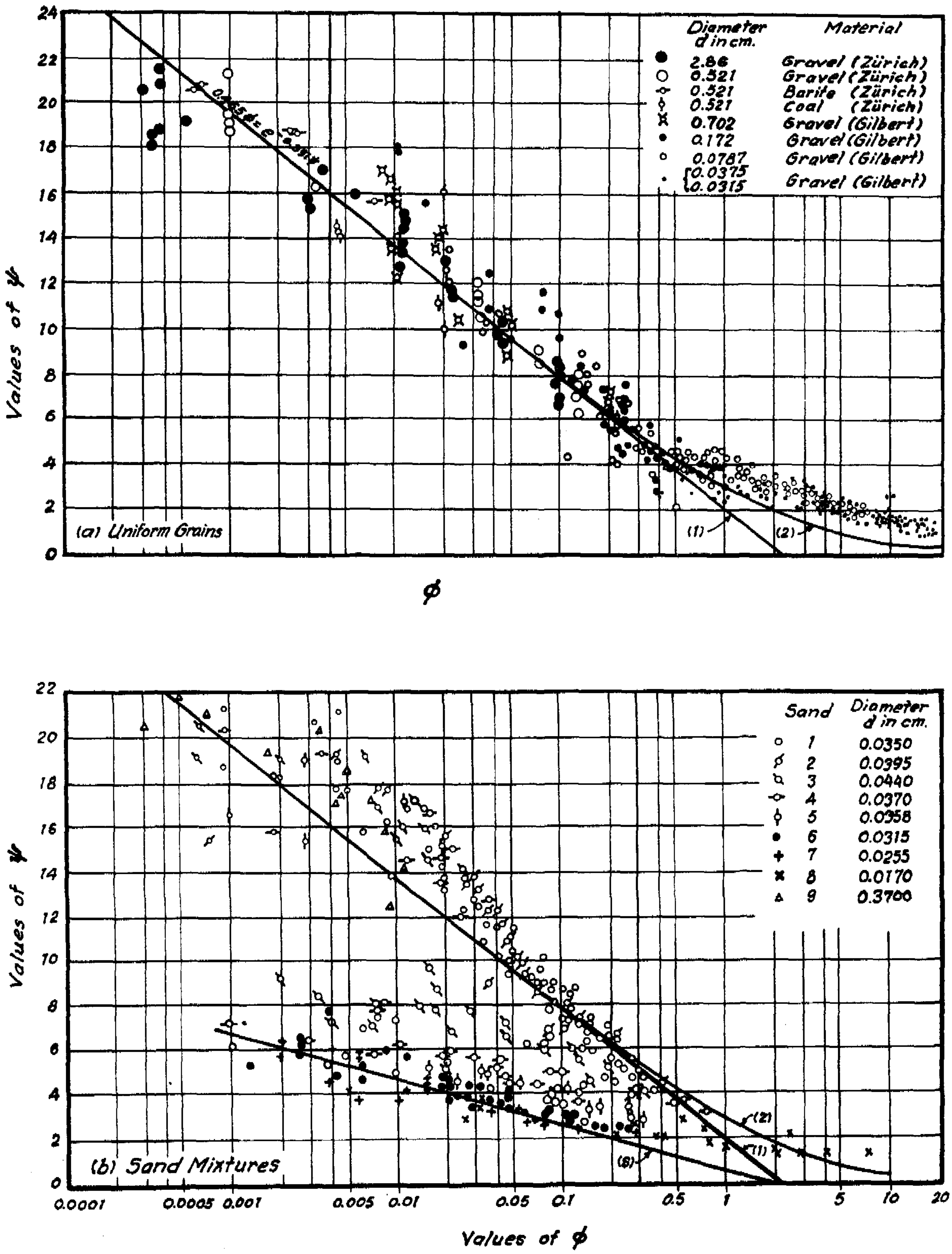

FIG. 13 GRAPH OF EINSTEIN'S ϕ–ψ BED-LOAD FORMULA

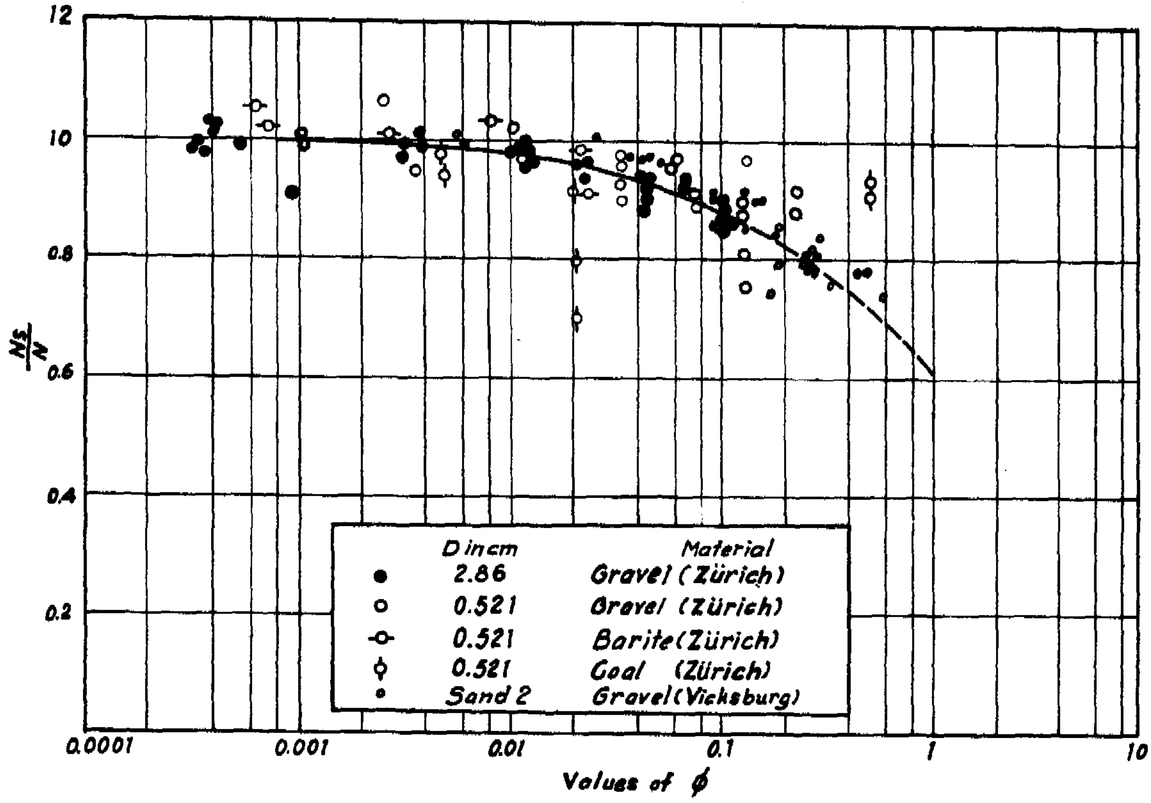

FIG. 16 RELATIVE BED ROUGHNESS PLOTTED AGAINST ϕ ACCORDING TO EINSTEIN

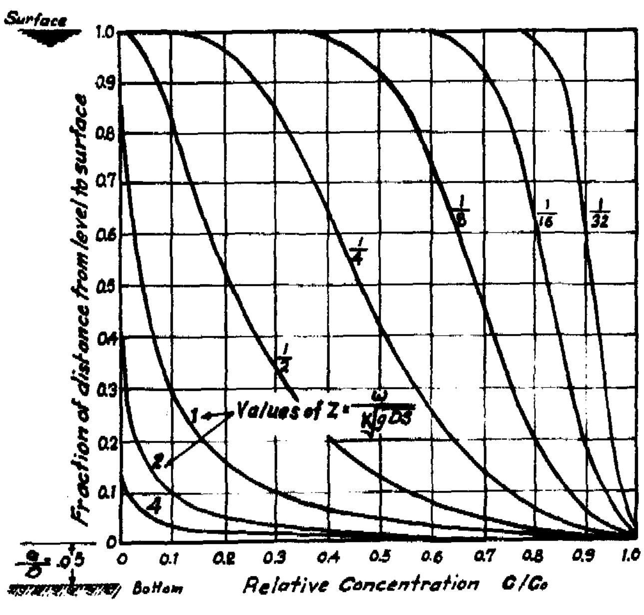

FIG. 17 GRAPH OF THEORETICAL SUSPENDED LOAD DISTRIBUTION

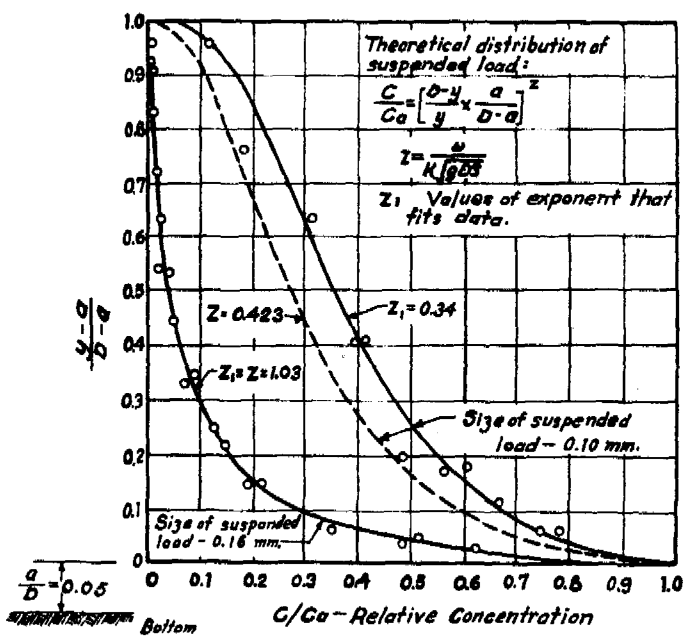

FIG. 18 COMPARISON OF THEORETICAL AND MEASURED SUSPENDED LOAD DISTRIBUTION FOR TWO SIZES OF SEDIMENT

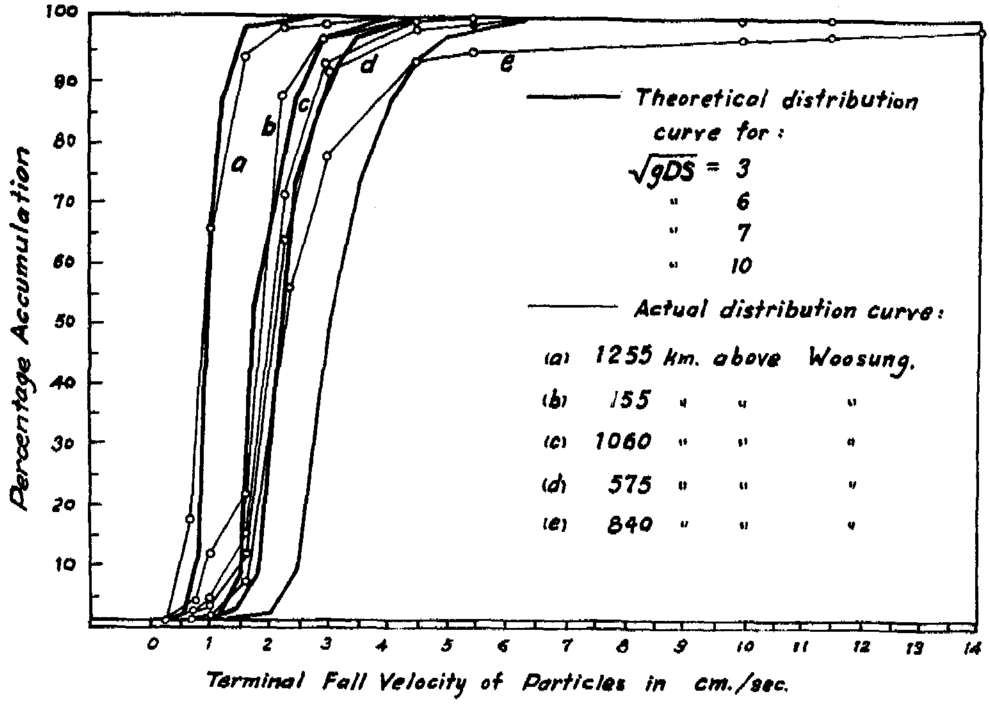

FIG. 19 COMPOSITION OF RIVER BED MATERIAL OF THE YANGTZE RIVER

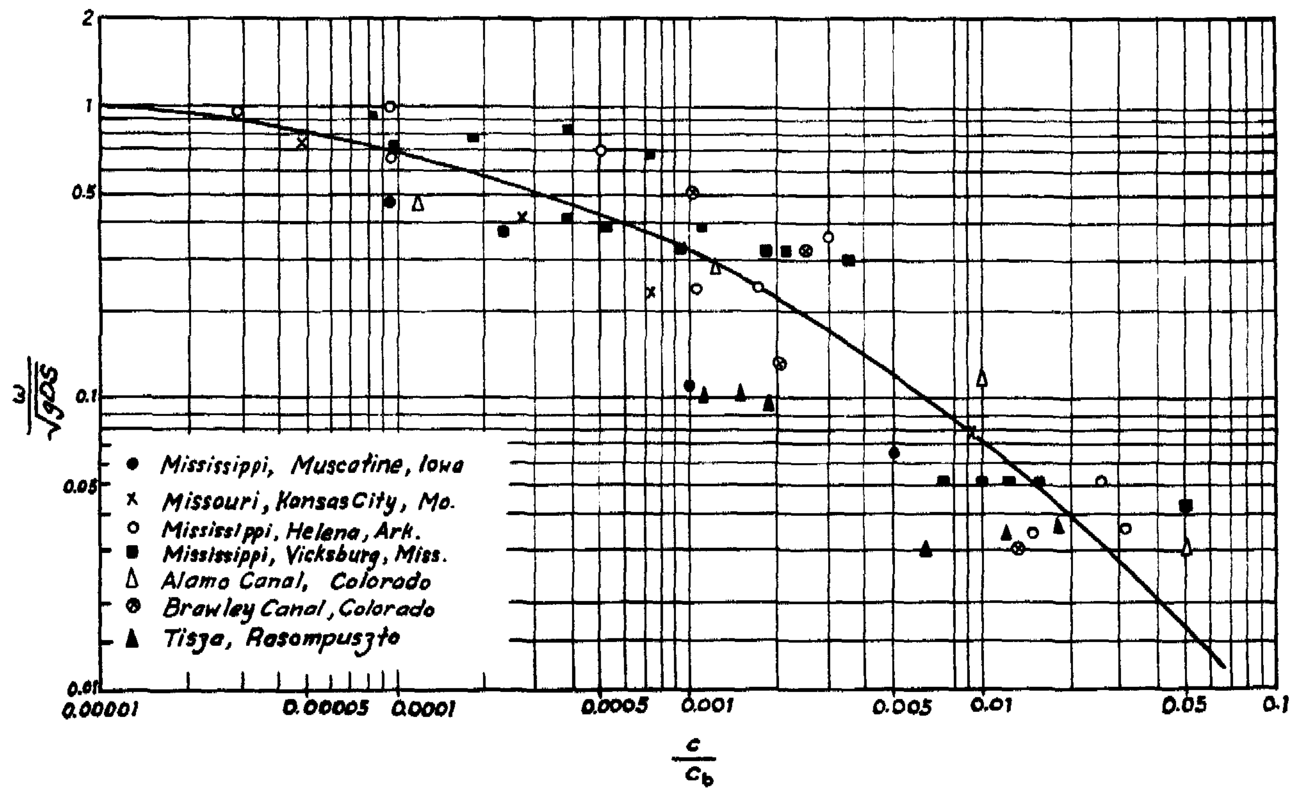

FIG. 20 THE $\frac{c}{c_b}$ FUNCTION OF LANE-KALINSKE

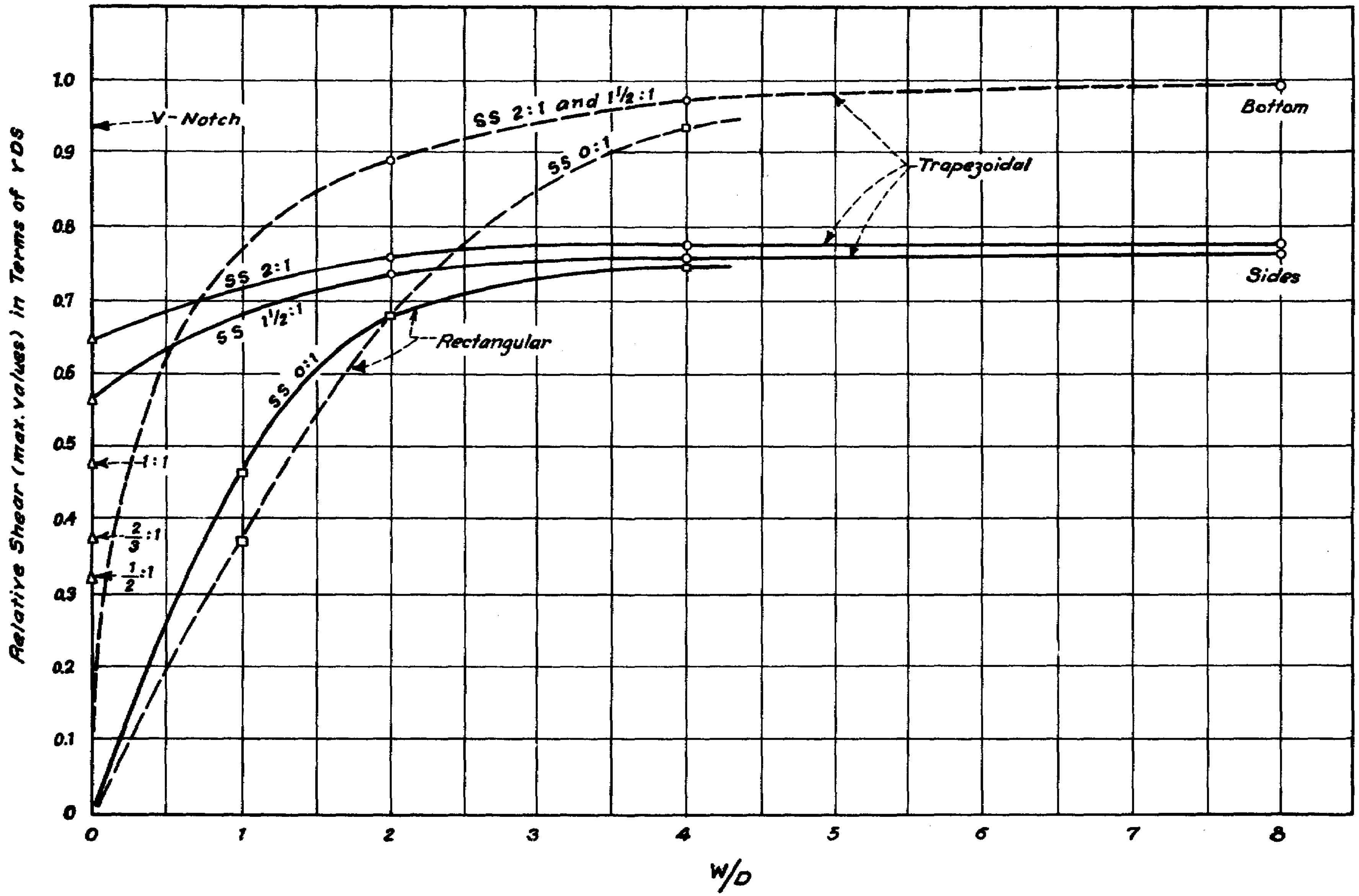

FIG. 21 THE MAXIMUM TRACTIVE FORCE ON BED AND SIDES EXPRESSED IN TERMS OF TRACTIVE FORCE FOR VARIOUS RATIOS OF BOTTOM WIDTH TO DEPTH

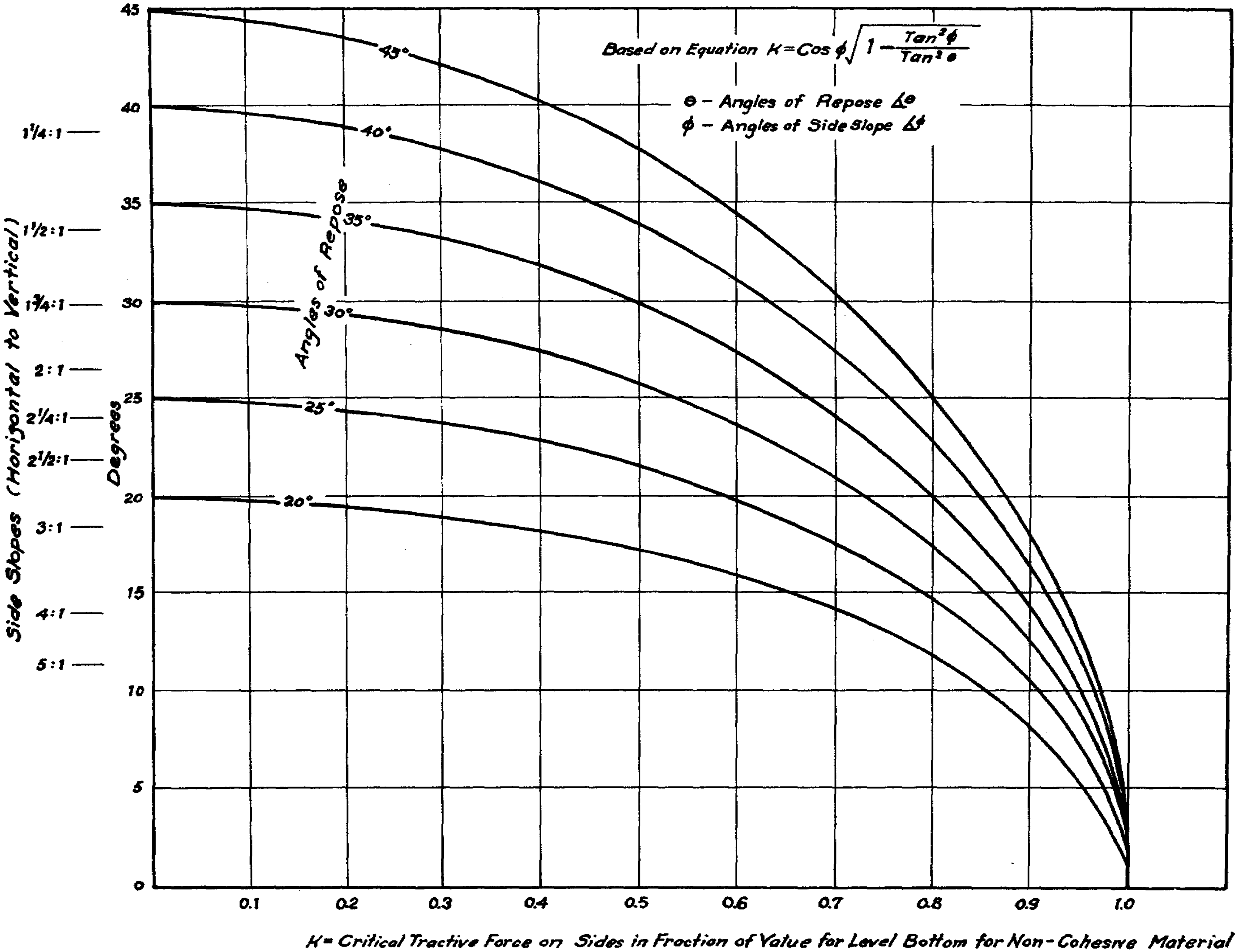

FIG. 22 TRACTIVE FORCE REQUIRED TO INITIATE MOTION PLOTTED AGAINST ANGLE OF REPOSE

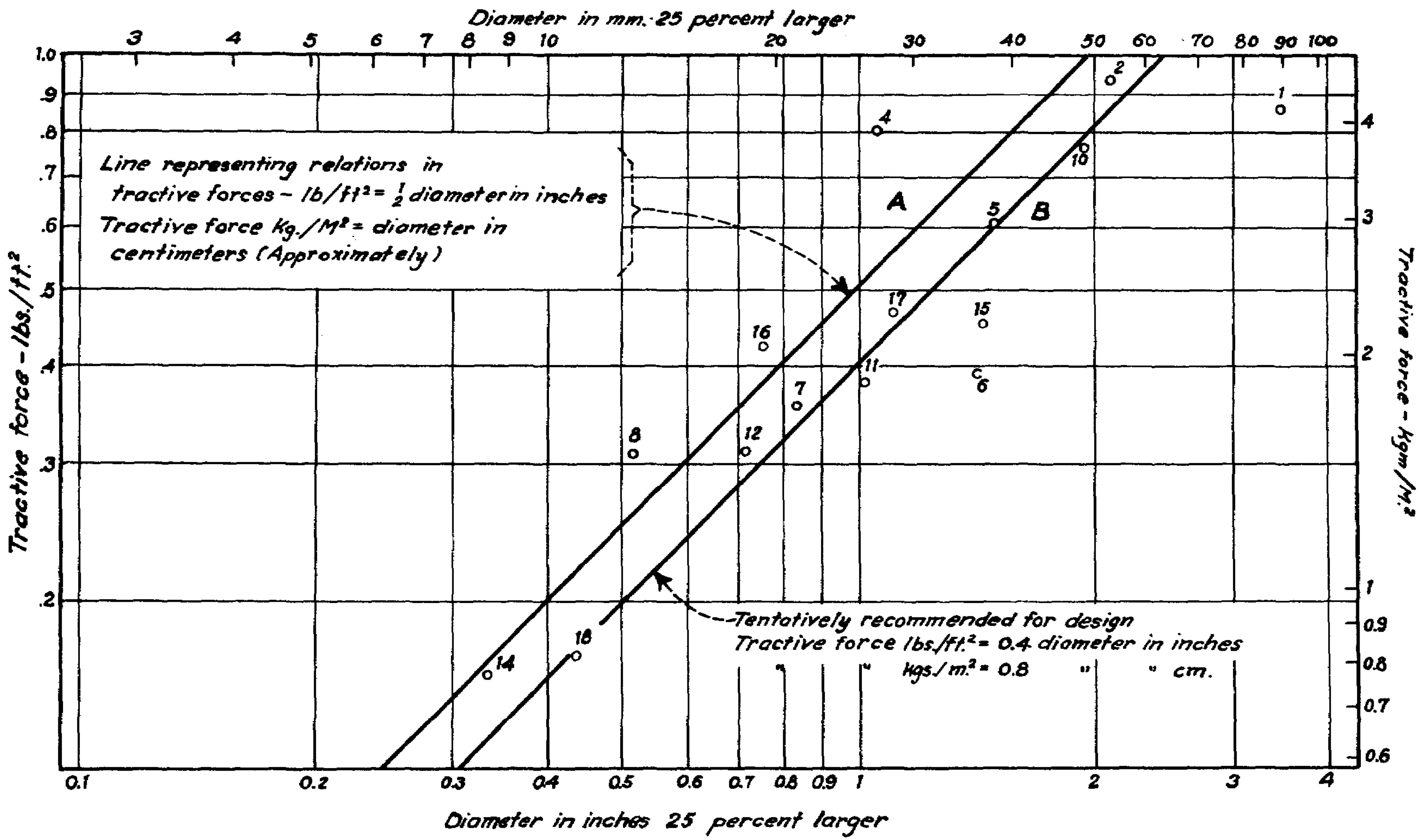

FIG. 23 CRITICAL TRACTIVE FORCE PLOTTED AGAINST MEAN DIAMETER OF PARTICLES

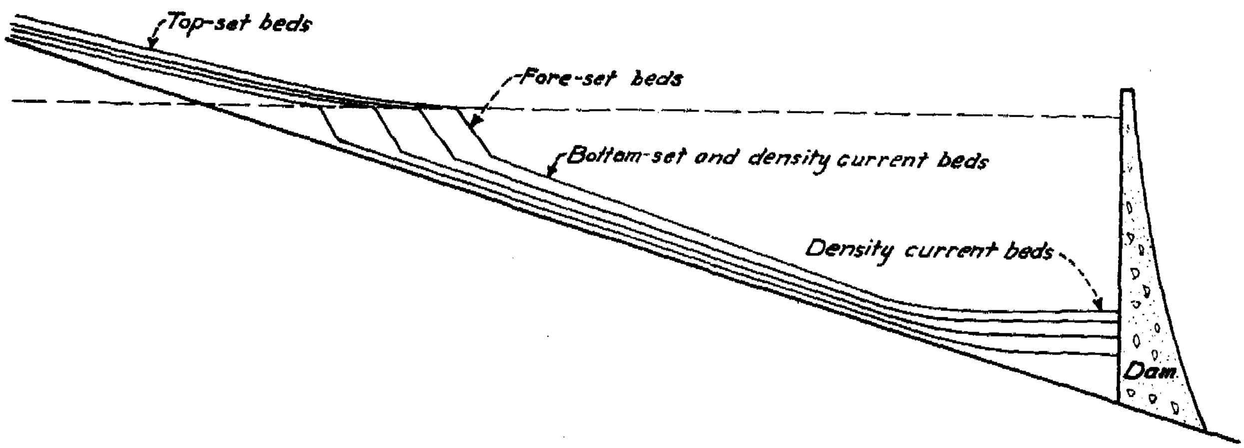

FIG. 24 A LONGITUDINAL SECTION THROUGH A RESERVOIR FORMED BY A DAM

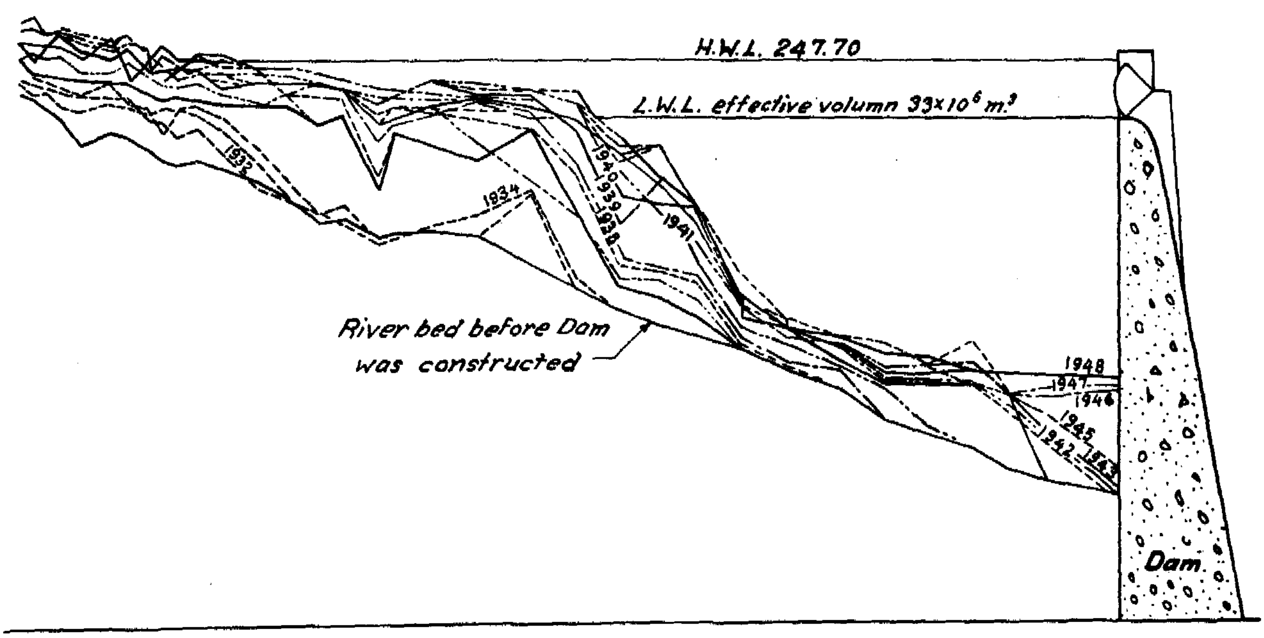

FIG. 25 BOTTOM OF THE SOYAMA RESERVOIR, JAPAN

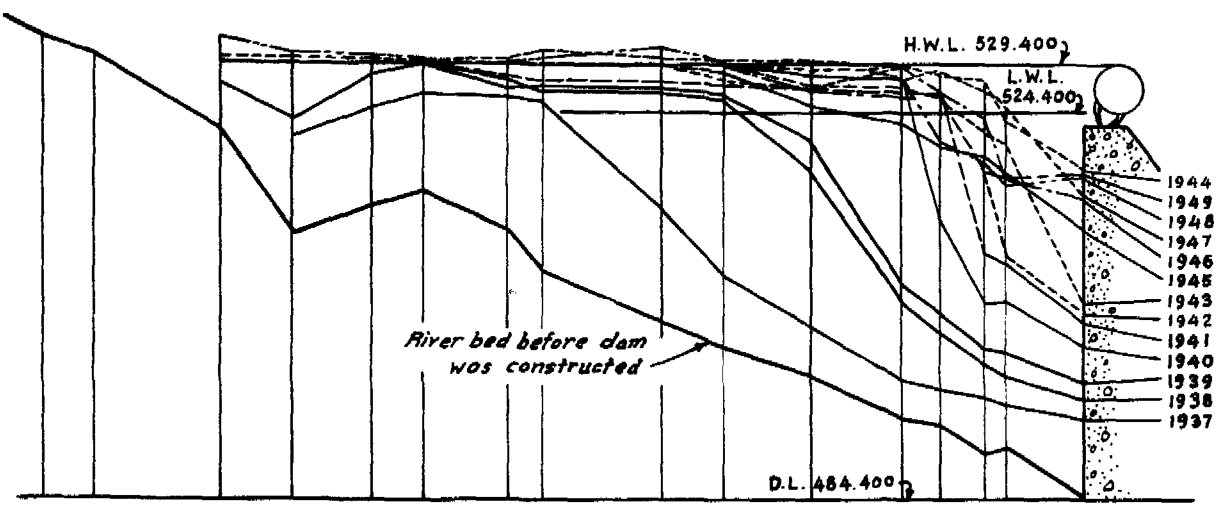

FIG. 26 BOTTOM OF THE KOYADAIRA RESERVOIR, JAPAN

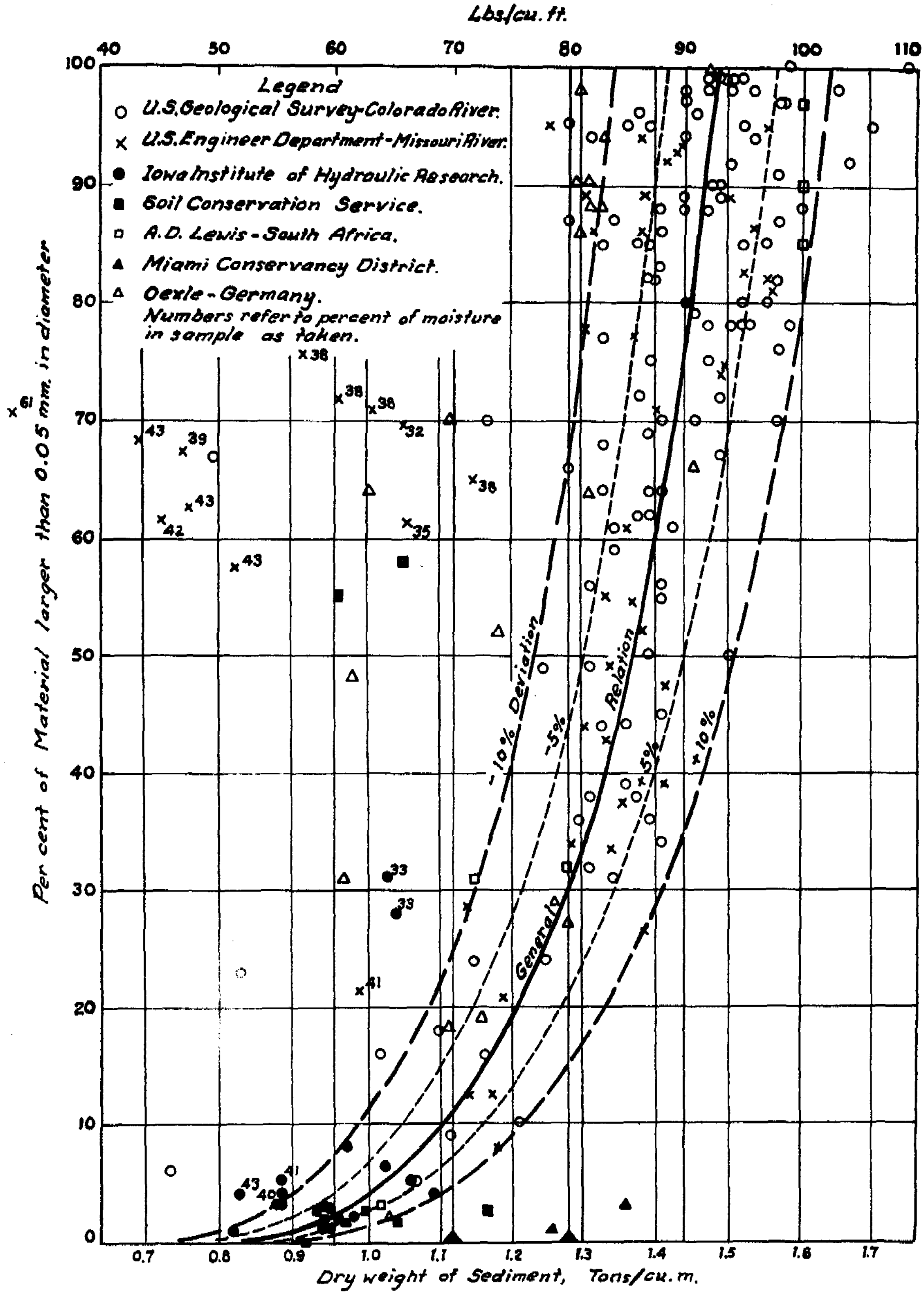

FIG. 27 RELATION OF UNIT WEIGHT OF DEPOSITED SEDIMENTS TO PER CENT OF SAND

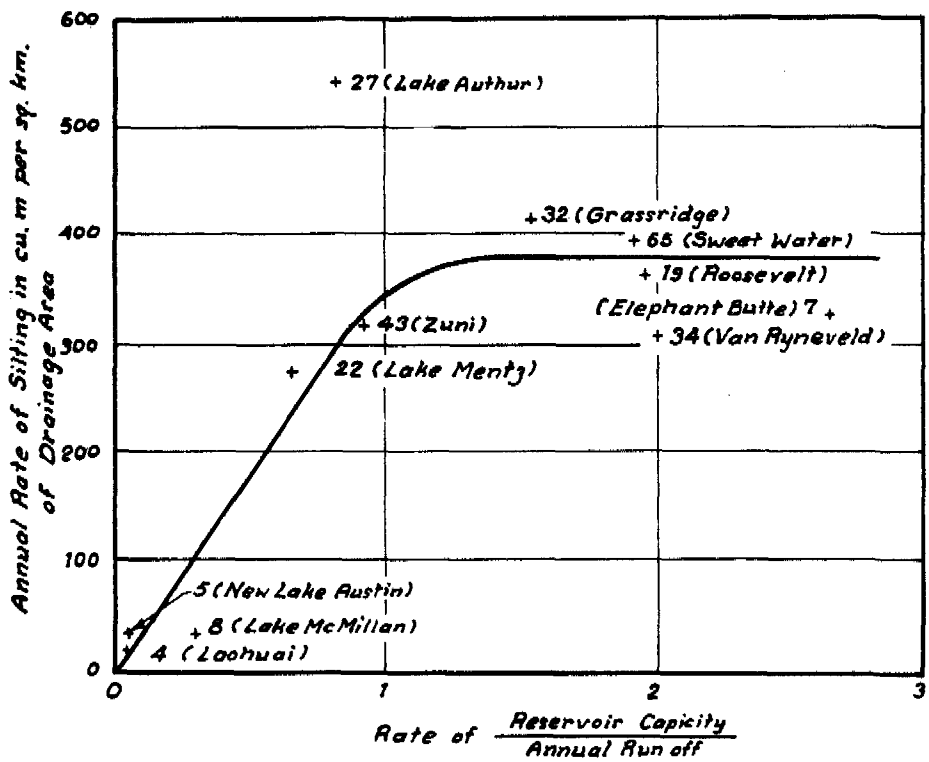

FIG. 28 RELATION BETWEEN ANNUAL RATE OF SILTING AND DRAINAGE AREA

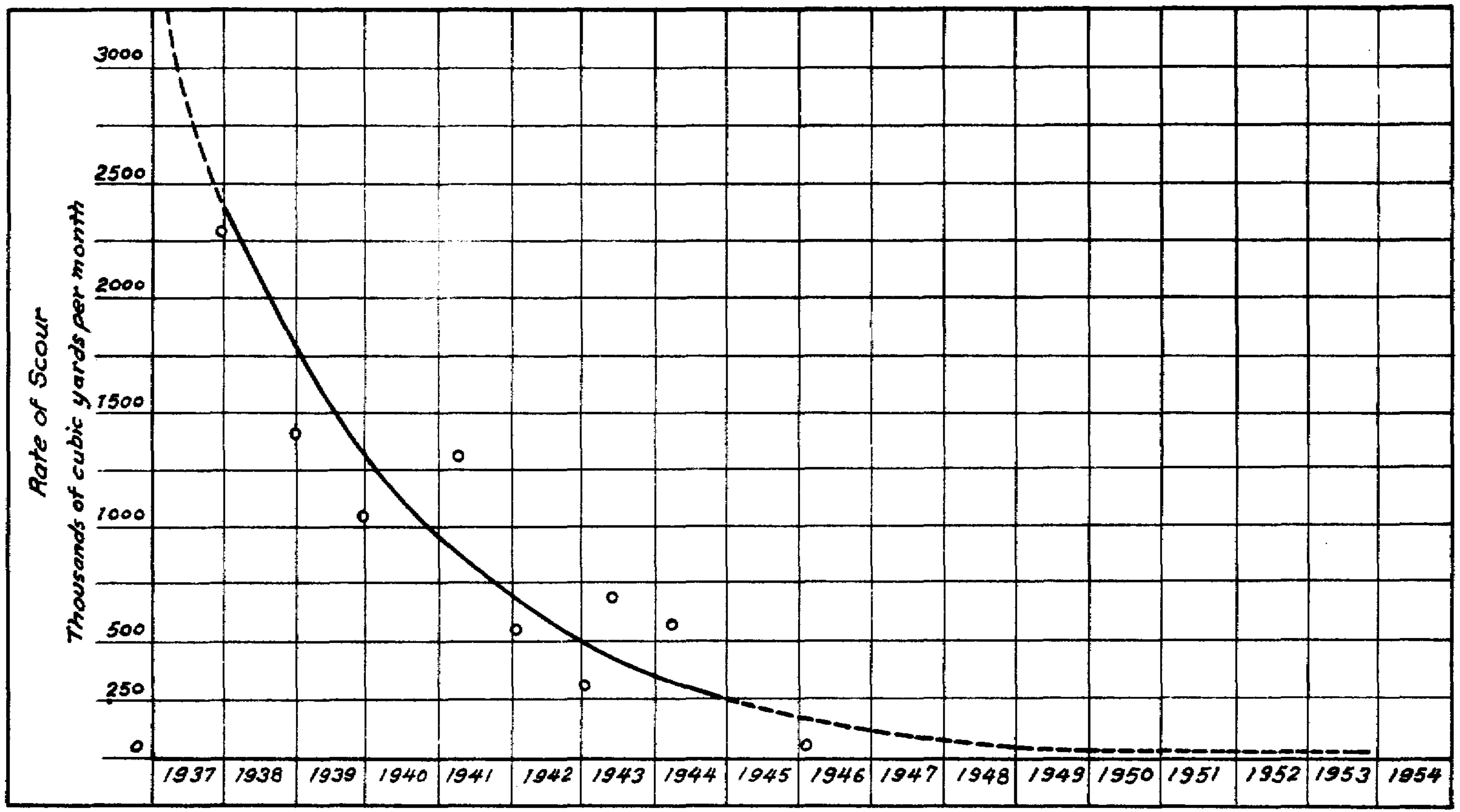

FIG. 29 AVERAGE ANNUAL RATE OF SEDIMENT TRANSPORTATION

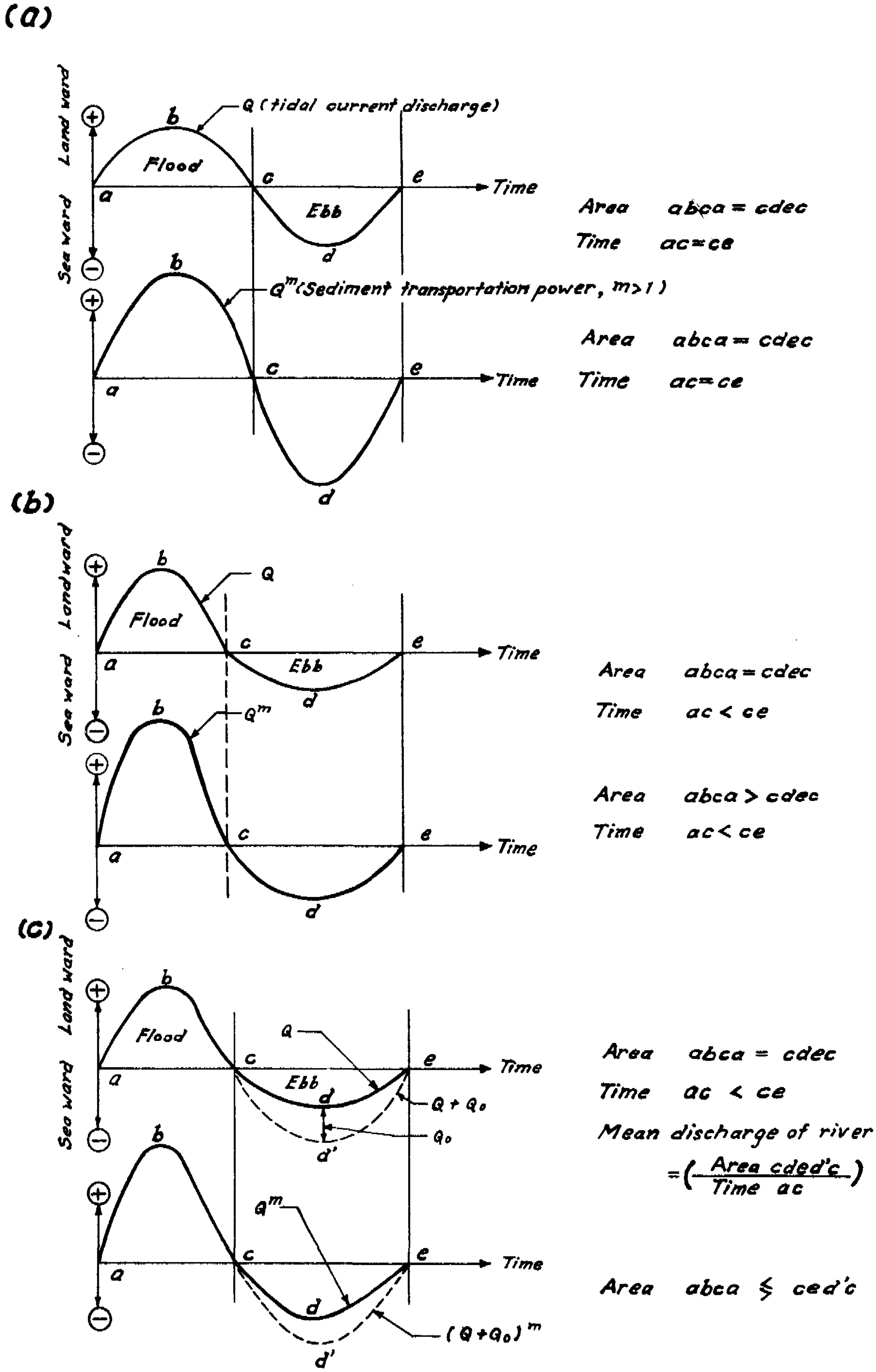

FIG.30 DISCHARGE AND SEDIMENT TRANSPORTATION POWER OF TIDAL CURRENT

UNITED NATIONS

PROCEEDINGS OF THE REGIONAL TECHNICAL CONFERENCE ON FLOOD CONTROL IN ASIA AND THE FAR EAST

This publication, which is the third number of the *Flood Control Series,* comprises the proceedings of the Technical Conferences on Flood Control organized by the United Nations Economic Commission for Asia and the Far East and held at New Delhi, India from 7 to 10 January 1951 and the thirty-two papers presented at this Conference by experts from member and associate member countries and by the various specialized agencies of the United Nations.

UN publication, sales No. 1953. II.F.1. Price US$3.00 or equivalent in other currencies. Available in English.

RIVER TRAINING AND BANK PROTECTION

This study, which is the fourth publication in the *Flood Control Series* prepared by the Bureau of Flood Control and Water Resources Development of the Economic Commission for Asia and the Far East, deals with the general theory and principles of river training and river bank protection. A description is given of the various methods of river training and bank protection that have been adopted in the different countries of Asia and the Far East as well as in Australia, Europe, New Zealand, and the United States of America. Recommendations are made on the basis of the comparison of different methods in use. A questionaire on river training and bank protection has been appended with a view to collecting further information on the subject.

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