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A contribution to the
International Hydrological
Programme, IHP – II Project
A. 2.6.1 Panel

Stevan Bruk, Rapporteur

Unesco, Paris

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KD 5933
RN: 5933
ISN 2131
LO: 214.0 85ME

February 1985

The Yugoslav National Committee for the International Hydrological Programme contributed to the printing of this book

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Printed by the Printing Department of the Jaroslav Černi Institute for the Development of Water Resources

P.O. Box 530, 11000 Belgrade, Yugoslavia

Preface

Although the total amount of water on earth is generally assumed to have remained virtually constant, the rapid growth of population, together with the extension of irrigated agriculture and industrial development, are stressing the quantity and quality aspects of the natural system. Because of the increasing problems, man has begun to realize that he can no longer follow a "use and discard" philosophy – either with water resources or any other natural resources. As a result, the need for a consistent policy of rational management of water resources has become evident.

Rational water management, however, should be founded upon a thorough understanding of water availability and movement. Thus, as a contribution to the solution of the world's water problems, Unesco, in 1965, began the first world-wide programme of studies of the hydrological cycle – the International Hydrological Decade (IHD). The research programme was complemented by a major effort in the field of hydrological education and training. The activities undertaken during the Decade proved to be of great interest and value to Member States. By the end of that period, a majority of Unesco's Member States had formed IHD National Committees to carry out relevant national activities and to participate in regional and international co-operation within the IHD programme. The knowledge of the world's water resources had substantially improved. Hydrology became widely recognized as an independent professional option and facilities for the training of hydrologists had been developed.

Conscious of the need to expand upon the efforts initiated during the International Hydrological Decade and, following the recommendations of Member States, Unesco, in 1975, launched a new long–term intergovernmental programme, the International Hydrological Programme (IHP), to follow the Decade.

Although the IHP is basically a scientific and educational programme, Unesco has been aware from the beginning of a need to direct its activities toward the practical solutions of the world's very real water resources problems. Accordingly, and in line with the recommendations of the 1977 United Nations Water Conference, the objectives of the International Hydrological Programme have been gradually expanded in order to cover not only hydrological processes considered in interrelationship with the environment and human activities, but also the scientific aspects of multipurpose utilization and conservation of water resources to meet the needs of economic and social development. Thus, while maintaining IHP's scientific concept, the objectives have shifted perceptibly towards a multidisciplinary approach to the assessment, planning, and rational management of water resources.

As part of Unesco's contribution to the objectives of the IHP, two regular publication series are issued: "Studies and Reports in Hydrology" and "Technical Papers in Hydrology". Occasionally, documents such as the present one are produced by special arrangements with IHP National Committees or other national entities associated with the production of the basic report and are made available for distribution from the IHP headquarters at Unesco, Paris.

Foreword

This technical report was prepared conforming to the decision of the Bureau of the Intergovernmental Council of the International Hydrological Program, which in 1981 appointed a rapporteur (S. Bruk) and two co-rapporteurs (Fan Jiahua nad H.E. Jobson), under Project IHP.II.A.2.6.1. In preparing the report, the rapporteurs were joined by the representatives of the International Commission on Continental Erosion (J. McManus) and of the International Training Center for Water Resources Management (CEFIGRE) (J. Evrard).

The terms of reference were "to review methods of computing sedimentation in lakes and reservoirs and to prepare a report on recent developments." The report was to provide a summary of recent developments in the study of sedimentation in lakes and reservoirs and in practical computation methods. It had to include recommendations on appropriate ways to prevent or reduce silting of reservoirs during their operation. Finally, the report had to try to identify further research needs and suggest possible action within the framework of the third phase of the IHP (1984–1989).

The contents of the report and the drafts of its chapters were discussed in the meetings in Paris, July 1981 and September, 1982. The six chapters were contributed by the following members of the group:

Chapter 1 – Technical and Economic Impact of Reservoir Sedimentation
by J. Evrard, Electricité de France, Beziers

Chapter 2 – Physical Processes of Reservoir Sedimentation
by J. McManus, University of Dundee, Dundee, U.K.

Chapter 3 – Field Measurements
by H.E. Jobson, U.S. Geological Survey, Mississippi, U.S.A.

Chapter 4 – Methods of Preserving Reservoir Capacity
by Fan Jiahua, Institute of Water Conservancy and Hydroelectric Power Research, Beijing, China

Chapter 5 – Prediction Methods
by S. Bruk, "Jaroslav Černi" Institute for Development of Water Resources, Belgrade, Yugoslavia

Chapter 6 – Conclusions and Recommendations
by writers of all chapters, collected and edited by S. Bruk

Mr. B.R. Payne from the International Atomic Energy Agency contributed to Chapter 3.

The drafts of all chapters were circulated among the members of the group and mutually commented. The manuscripts were reviewed by Dr. McManus for English editing and Mr. H.E. Jobson took care of bringing the report into a camera–ready form.

Mr. Zhang Youshi, Senior Program Specialist in the Department of Water Sciences of Unesco acted as Technical Secretary to the project and contributed to the coordination of the efforts.

International organizations, such as the World Meterological Organization, International Atomic Energy Agency, International Commission on Irrigation and Drainage, International Association of Scientific Hydrology and its Commission on Continental Erosion, as well as the International Training Center for Water Resources Management – CEFIGRE, were kept informed about the progress of the project.

The contents of the report were discussed at the first meeting of the rapporteurs at Unesco, Paris, between June 29 and July 1, 1981, and later revised at the 2nd meeting also held at Unesco, Paris, from September 20–24, 1982. The report was completed and submitted in a preliminary form for discussion at a special session of the 2nd International Symposium on River Sedimentation, Nanjing, October 11–16, 1983.

In the final report, the authors responded to the comments and suggestions made during the discussions at Nanjing. A proposal of the follow–up activities worked out at the meeting in Nanjing is included in the last chapter of the present report.

Belgrade, February, 1985 Stevan Bruk, Rapporteur

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Introduction

SCOPE OF THE REPORT

The computation of reservoir sedimentation is in the present report considered as a tool of engineering predictions in the planning, design and operational phases of reservoir projects. Being oriented towards definite engineering purposes, the computations thus require a clear statement of the objectives of predictions, i.e., of what the technical and economical importance of the problem is, what physical phenomena should be modeled, what engineering measures should be planned based on the predictions and what kind of field measurements can be made in support of the computations.

Hence the scope of the report is somewhat wider than the title implies: in addition to only reviewing computational methods, the report also considers all the other aspects of reservoir sedimentation which are essential from the engineering point of view. Thus, the technical and socio-economic impacts of reservoir sedimentation are discussed in Chapter 1; the physical phenomena related to sedimentation in lakes and reservoirs are described in Chapter 2; measurement methods and instrumentation are reviewed in Chapter 3; the methods of preserving the reservoir capacity and recovering the lost storage are dealt with in Chapter 4 while the principal methods of engineering predictions are in the background of the foregoing chapters and are given in Chapter 5. Finally, the principal findings of the report are summarized in Chapter 6 which also contains a proposal of the follow-up activities for the 3rd phase of the International Hydrological Programme.

Due to the limited available time and resources, the report could not embrace all the diversity of sedimentation problems in lakes and reservoirs. As the engineering impact of sedimentation is by far more important in manmade reservoirs than in natural lakes, more attention was given to reservoirs than lakes in this report. Similarly, the report is mainly concerned with deep reservoirs rather than with shallow water impoundments which feature many physical and ecological processes not discussed in the present report. It is recommended however, that shallow impoundments could be the subject of a special technical report in the next phase of the IHP.

THE ADOPTED APPROACH

Aware of the pre-set limitations, the authors adopted a realistic approach in compiling the report based on the following principles:

- use of all the information available to the authors when writing their contribution without attempting to produce and use a virtually complete and all-embracing list of references.

  Considerable assistance in enlarging the data base of the report was given by the documentation service of Electricite de France. The documentary material collected by CEFIGRE was also put at the disposal of the authors of the report.

- shared responsibility for all parts of the report by all contributors.

  The principle was attained by discussing the chapters at meetings and by circulating all the contributions among the authors before they were printed.

  The last revision of the draft report was made at the 2nd International Symposium on River Sedimentation (October 11–14, 1983, Nanjing, China).

- Observation of the deadline set for the preliminary report (October 1983) and final report (Decembr 1983).

  The authors felt that the topic of reservoir sedimentation requires urgent action and that the report needs to serve as a background for further action rather than to aim at being the latest word on the subject. It was thought more important to have the report ready on time than to expand and polish its contents on account of delaying its publication.

POINTS EMPHASIZED IN THE CHAPTERS OF THE REPORT

The following is a brief comment on the points emphasized in the various chapters of the report.

Chapter 1 "Technical and Economic Impacts of Reservoir Sedimentation" (Author J. Evrard)

Emphasis was laid on the completeness of the review of all the impacts, for the benefit of planners, designers and users of reservoirs.

Chapter 2 "Physical Processes of Reservoir Sedimentation" (Author J. McManus)

Emphasis was laid on the qualitative description and understanding of the phenomena encountered in lakes and reservoirs relative to sedimentation processes.

Research needs were underlined with respect to the phenomena which still require a better understanding.

Chapter 3 "Field Measurements"
(Author H.E. Jobson with a contribution from B.R. Payne).

New techniques of measurement specific for reservoirs, such as are surveying, determining of the density of deposits, dating of deposits, etc., were emphasized whereas the techniques of standard sediment measurements were only mentioned in short.

Chapter 4 "Methods of Preserving Reservoir Capacity"
(Author Fan Jiahua)

The various methods used all over the world were illustrated by a large number of case studies, some of which were described in an international publication for the first time.

Chapter 5 "Prediction Methods"
(Author S. Bruk)

Emphasis was placed on stating the objectives and philosophy of prediction, also giving a short review of the standard methods. A reference to new developments was also given.

In addition to the modeling of reservoir sedimentation and river bed degradation downstream of dams, methods of computing the storage recovery and preservation were also reviewed.

Chapter 6 "Conclusions and Recommendations"
(Edited by S. Bruk)
The conclusions of the chapters of the report are summarized and the follow-up activities recommended. The recommendations reflect a broader view of reservoir engineering in which reservoir sedimentation problems are linked to water quality problems. The need for a specific review of shallow reservoir problems is also underlined. review of shallow reservoir problems is also underlined

Chapter 1
Technical and Economic Impact of Reservoir Sedimentation

The management of water resources often requires the construction of dams, in order to control the irregularity of river discharges. The construction of a reservoir greatly disturbs the social and economic environment, even before its benefits are felt.

Depending upon the purpose of the dam, the benefits will vary in place and time. So, for instance, the benefits drawn from a hydroelectric project will be felt immediately after its implementation, but not at the site of the dam due to the transmission of the electric current into the power system. On the contrary, the benefits of an irrigation project will be felt locally, but only after a lapse of time needed to activate the agricultural potential of the irrigated land.

Any reservoir on a sediment carrying river will gradually become silted up, even though this process may take a long time. It is, therefore, most important to take care that the silting up of a reservoir does not occur before the benefits from it are fully achieved. It would be a gross error of the decisionmakers if the sedimentation of the reservoir was not foreseen and if no measures were taken in order to check it, or to reduce its effects to an acceptable level. Neglect in this respect would cost dearly in the future, since the available water resources are limited as are the economically suitable reservoir sites within a basin.

1.1 PHYSICAL AND BIOLOGICAL IMPACTS

The construction of a dam in the river valley causes:

- A modification of the regime of river discharges
- A transformation of river channel morphology in the zone of the reservoir
- A significant change of the conditions of sediment transport.

Downstream discharges may be reduced if a part of the flow is diverted by the project.

The unity of the river basin does not allow modification of one reach of the river without disturbing the basin as a whole to some degree, particularly with regard to sediment transport and its effects.

1.1.1 Influence on the Reservoir

By impounding a river, the cross sections available for the flow increase and the flow velocity decreases; which leads to a decrease of the sediment transport capacity. This causes the deposition of sediments, the consequences of which depend on the size, shape and location of the deposits. The mechanism of this process is described in Chapter 2. It can be summarized as follows:

The coarser particles are deposited first, building up an underwater delta near the upstream end of the reservoir, the configuration and progression of which depend on the regime of flow and the variation of water levels in the reservoir. This delta causes a rise of the original river bed and, in addition to the reduction of the active storage, it may cause a reduction of the clearance below bridges which may represent a danger to navigation. It also causes an increase of water levels during floods, which may lead to the submergence of the riverside land, particularly when the emerging sand banks are recultivated or overgrown by shrubbery.

A further consequence of delta formation is the raising of the water table, which results in the appearance of marshes.

The finer sediments are carried into the reservoir and settle on the reservoir bottom depending on the flow velocity and wind-induced or thermal currents. These fine materials will be spread over the littoral and sublittoral zone from which the silt and mud will be carried progressively to the dam where it accumulates and, in some cases, creates a real mud lake.

The deposited fine materials may affect both the active and the dead storage.

In the littoral areas there will be a marked rise of the bottom of creeks and shallows, which may lead to the development of aquatic growths. These may, in turn, accelerate the consolidation of the deposited silt and mud. If these emerge over prolonged periods of time, they may dry up and become eroded by wind.

In the deep sublittoral zones, the deposited silt will progressively consolidate, gradually filling the dead storage, except when sediment flushing is practiced.

The consequences of the deposition depend on the position of the river inflow in relation to the structure. It is obvious that the inflow of a tributary close to a water intake must be carefully controlled.

The effects of bank erosion caused by wave action and reservoir level fluctuations must also be taken into account, in addition to the transport of sediments.

1.1.2 Influences on the River Reach Upstream From the Reservoir

The sediment deposits in the reservoir will cause a raising of the river bed, progressing in the upstream direction. This will have similar effects as the already mentioned delta formation at the upstream end of the reservoir: braiding of the river channel, submersion of the riverside land, reduction of the clearance below bridges, etc.

On the other hand, if extensive sediment control methods are applied in the catchment, such as soil conservation, building of sediment traps, etc., the increased capacity to carry sediments downstream from the protection works may cause scouring. This will not always be in balance with the effects of the increased water levels in the reservoir.

1.1.3 Influences on the River Downstream From the Dam

The effects of a large dam are strongly felt on the river reach downstream from the dam. This is because the dam traps a more or less substantial part of the sediments, and modifies the grain size distribution of the released sediments by retaining most of the coarse particles in the reservoir.

The reservoir also modifies the regime of the flow downstream from the dam, even if no discharges are diverted by the project (i.e., the annual discharge remains unchanged). In most cases, medium and low floods are reduced by the reservoir. This modification is even stronger if part of the flow is diverted by the dam.

(A) Influences on River Morphology

The modified discharges downstream of the dam of water with reduced sediment content will scour the river bed. The bed load transport will increase in the downstream direction until an equilibrium between the transport capacity and sediment load is reached. The river bed will degrade downstream from the dam, unless limited by rock sills in the bed. The deeper river channel will be favorable for the passage of floods; but, on the other hand, degradation of the river bed downstream from a dam will also have negative effects:

- The deepening of the river channel may affect the water table in the valley either by lowering it if the groundwater flows towards the river, or by reducing the supply if the groundwater is fed from the river.
- It may lead to the pollution of groundwater or increase the salinity of the soil in the vicinity of an estuary.
- It can contribute to the undermining of structures and river banks.

If a part of the discharge is diverted by the dam, the eroding effects will be reduced, since the released discharges will be lower.

The released discharges may be saturated with sediments if an important part of the sediment is also flushed into the river downstream from the dam. The eroding capacity of the river is then reduced and deposits of sediments may appear below the dam. The resulting increase of the bed level may hamper the passage of floods and may cause the channel to meander.

The effects of the project on the river morphology should be studied by means of mathematical models, by which the downstream watercourse can also be represented. By using such models the necessary protection works and operational rules may be determined.

The reduction of low and medium floods by the reservoir and the trapping of most of the sediments prevents the possibility of warping, i.e., of conducting fertile silt onto the land in the river valley. It must be noted that, apart from fertility, the deposition of silt in the valley is useful because it may counteract the effects of eolian erosion, enhanced by cultivation of the land, and thus favors the conservation of the soil.

(B) Influence of Torrential Tributaries

At the confluences of torrential tributaries, cones of coarse sediment appear, which are then eroded during the floods of the main river. The reduction of low and medium floods by the reservoir may stop this action. The torrential cone may impede the flow in the river, and a high flood then may push the stream towards the opposite bank, and even cause the submersion of upstream land due to the obstruction of the channel. When the obstacle breaks, high discharges are suddenly released causing damage downstream from the obstruction.

These difficulties can be overcome by erosion control in the catchments of the tributaries and by the construction of sediment traps. Until the torrent becomes controlled by these measures, the confluence of the torrent should be guided in the downstream direction by a suitable system of dykes.

(C) Influences of Gravel Quarries

The partial or near total interception of coarse sediments by the reservoir will prevent the renewal of gravel layers which are exploited as building material.

It is, therefore, important to develop a policy in managing the quarries so as to prevent premature exhaustion of the resources. It would be advisable to encourage extraction upstream from the dam, in the area of the delta near the upstream end of the reservoir. Excessive excavation of the gravel layers could cause a lowering of the water table, and uncontrolled gravel extraction might cause serious damage to the stability of the channel during high floods.

When the released discharges are reduced by diversion, the problems may be aggravated because the easier access to the flood plain encourages the opening of gravel quarries.

(D) Influence on the Estuary

The reduction of sediment transport disrupts the balance of sediments at the river mouth, particularly in a delta region. Coastal erosion by currents and wave action is no longer compensated by the deposition of sediments brought by the river, and fertile agricultural land may recede or completely disappear.

High floods may abruptly complete the action of the currents in underscouring the coast, particularly if the reservoir is near the estuary; so that the unsaturated flow during the floods has a significant potential for erosion. The inflow of flood discharges may also lead to the development of recirculating currents along the shore which may strenghten the littoral transport of sediments.

If these effects are found intolerable, it is possible to remedy them by appropriate protective structures, such as dykes, breakwaters, groins, etc. These works, however, are very costly and should be designed by experts, on the basis of mathematical or physical models, after having their objectives clearly defined.

(E) Effects of Sediment Ejection

Accumulated mud is ejected into the river downstream from the dam in conjunction with operations carried out for the following reasons:

- Maintenance: regular emptying of the reservoir (since underwater inspection is not always possible), repairs, periodic testing of the gates . . .
- Reservoir operation: release of floods . . .
- Economy: preservation of the storage capacity by flushing of sediments, application of particular operational rules for the evacuation of mud, dredging, siphoning . . .

The downstream ejection of mud is inevitable, in any case, even if it is not wanted. At the end of the operational cycle, when the storage becomes empty, the river cuts its bed in the muddy deposits, until is has reached the level of the original river bed. The mud terraces are thus drained and large sections collapse. It should be noted that an emerged bank collapses at a lesser angle than an immersed one. Hence, the emptying of a reservoir will increase the slides. In the case of deliberate release of mud, the consequences can be attenuated by judicious mixing of surface and bottom layers of water.

The concentration of sediment in the released water can cause irreparable damage. The released sediments will be deposited selectively, the coarsest particles near the dam, and the finer particles farther downstream to form sand banks, silt banks and, finally, silt shoals. The deposition of these sediments will modify the slope of the river bed and obstruct the flow. This may disturb the passage of floods and cause submersion of riverside land. The modification of grain size distributions of sediments in the river bed may hamper gravel extraction. Siltation of the river bed may clog the river bed and hamper infiltration into the ground, which can cause serious damage to agriculture. Finally, the release of mud could be detrimental to the environment. This will be discussed in the following sections.

For all these reasons, it is necessary to study the conditions of mud evacuation. These studies should include the following:

- The geology of the basin in order to determine the characteristics of the sediments. A comparison with similar sites may facilitate the estimation of the rheological properties of the sediment deposits.
- The morphology of the river and the operational aspects of the reservoir.

1.1.4 Influence of Sediments on the Aquatic Biocenosis

Sediment is one of the factors of the aquatic biocenosis, which is governed, particularly by the energy flux transmitted by solar radiation (determining the photosynthetic activity) and by the recycling of organic matter. The penetration of solar radiation into the water depends on the clarity of the water; therefore, also on the concentration of sediment suspension.

The cycling of matter is related, inter alia, to the nature of transported materials and sediment deposits in the reservoir. These latter are mainly composed of products of soil erosion in the catchment to which are added various substances resulting from human activity.

The substances originate from:

- Surface runoff and leaching of irrigated soil through irrigation drains in the case of excessive gravity irrigation; they include organic fertilizers such as compost and manure, but consist more frequently of chemical fertilizers with a nitrogen or phosphorous base and, invariably, include pesticides or toxic substances such as weed killers, insecticides or fungicides.
- Untreated or insufficiently treated urban and industrial waste water containing not only organic matter, but also toxic salts of heavy metals such as chromium, cadmium and mercury.

Synergic reactions between certain substances are not to be excluded and a selective accumulation of some of these substances has been observed in the sediments. They can be found in increasing concentration throughout the trophic cycle. As a result, certain toxic substances, practically undetectable in water, can be found in sediments in sufficient quantities to cause serious harm to higher organisms.

In addition to their direct effects, these substances may have an indirect effect on the biomass. They can affect biological activity by acting on the "chemical telemediators"; these latter substances are synthesized by living organisms, and seem to condition their behavior in respect to the essential functions of nutrition, reproduction and mobility.

In conclusion, a dam is not "per se" an active polluting factor. However, by trapping sediments and accompanying nutrients, it can bring out certain phenomena, the impact of which should be determined by appropriate studies, such as:

- A hydrological inventory or assessment of the river.
- Detection of the biotic pollution indicators and, where applicable, toxic substances which accumulate within benthic or pelagic trophic cycles.
- A prediction based on a numerical model of the evolution of the ecosystem; such models are based on the laws governing biological processes and fluid mechanics and seem to be very promising, despite the delicacy of their development.

Such studies, in conjunction with hydraulic models of sedimentation, can improve the design of the reservoir, eliminating or attenuating its damaging effects.

(A) Influence on the Biocenosis in the Reservoir

Suspended matter restricts the chlorophyll activity; this leads to a diminution in the production of phytoplankton and impoverishment of the trophic cycle.

In the littoral zone where the water is penetrated by solar radiation, sediment forms a substratum permitting the growth of aquatic vegetation; in shallow areas this vegetation consists of rushes, reeds, etc., while algae grow in deeper zones to the extent permitted by climatic conditions and variations of the water level. The higher the content of nutrients in the reservoir water, the more intensive the growth.

The reeds and the underwater fields of algae provide shelter for the benthos, thus contributing to the development of a favorable biotope for fish breeding.

In the deep sublittoral zone where little or no light penetrates, vegetation disappears and the deposits accumulate, generally showing a seasonal stratification. Organic matter, particularly rapid decomposition of algae and plants, causes an oxygen deficit leading to anaerobic conditions. The bacteriological mineralization, which is particularly intensive at the water-mud interface, is arrested; putrid fermentation then takes place on the reservoir bottom which become lifeless.

Furthermore, sedimentation buries the spawning grounds and hampers the reproduction of those species of fish which spawn on gravel banks.

The passage from the lotic to the lentic stage causes, in the case of loss in the trophic value, a profound change in the living communities and, consequently, in the fish population; this deterioration, as far as productivity and quality are concerned, can sometimes be corrected by appropriate management and breeding.

(B) Influence on the Biocenosis Upstream From the Reservoir

The effects of reservoir sedimentation on the upstream reach of the river are generally limited.

The raising of the river bed at the upstream end of the reservoir usually leads to the formation of gravel banks and causes braiding of the channel, creating an area favorable to spawning. However, since large spawning grounds in the reservoir become covered with mud and the water levels fluctuate in the reservoir, some species of fish migrate towards the tributaries, causing temporary overpopulation in their lower reaches. This adversely affects reproduction rates by impoverishing the food supply and encouraging predation.

(C) Influences of the Biocenoisis in the River Downstream From the Dam

Whatever precautions are taken, the reservoir accumulates an appreciable portion of the products of erosion and, due to the process of concentration, a relatively more important part of the nutrients. The impact on the river downstream from the dam depends upon the operation of the reservoir.

In normal operation, when water is released through the intake structures, it contains only dissolved solids and virtually no sediments in suspension. This leads to an impoverishment of the biomass in the downstream water course and, where applicable, its estuary. This situation may be aggravated if part of the discharge is diverted by the project.

The reduction of the biomass leads to a decrease of the productivity of fish breeding, although in some cases, this can be compensated by the supply of young fish.

In exceptional situations when the bottom outlets are opened to evacuate floods, flush sediments, or to empty the reservoir, a substantial part of the sediment deposits becomes eroded and high sediment concentrations appear in the released water, much higher than in natural conditions. The effect of the turbid water on the biocenosis depends upon the physicochemical characteristics of the released water and the conditions in which the release is carried out.

When the ejected mud does not contain toxic substances, the pollution is only of a mechanical nature; but, it can cause the death of the more sensitive species of fish due to the obstruction of gills and can destroy spawn by clogging the gravel banks with deposited silt. However, the effect of this pollution is only temporary, without lasting consequences.

On the other hand, putrid mud with a high content of harmful substances is far more lethal to the fish. The benthos can be decimated and a long period of imbalance may ensue.

The consequences of sediment evacuation are especially bad when it is carried out without regard to the sediment transport capacity of the downstream river reach: banks and shoals of toxic mud may appear, sterilizing large stretches of the river.

The effects of releasing sediments must, therefore, be carefully studied beforehand. The frequency of such operations must be planned, taking into account the biological cycle of the fauna so as to attenuate the consequences as much as possible. Such studies should encompass the survey of the downstream river reach, including the estuary, where necessary; because the sudden arrival of large quantities of mud can have serious consequences on the marine environment.

Finally, it should be underlined that the release of mud into downstream river reaches cannot be avoided, any more than the erosion of soil in the catchment itself can be prevented. Society must compromise between the preservation of man-made wealth and the natural legacy it has inherited. As regards the the project itself, acceptable operational methods have to be devised which will cause the least disturbance to the life in the river. Where necessary, the project must accept the additional cost of remedial measures.

1.2 IMPACT ON THE SAFETY OF THE PROJECT

Sediment deposits may affect the safety of the dam.

1.2.1 Influences of Sediment Deposits on the Bottom Outlets

One of the most important hazards of sedimentation is the choking of the bottom outlets by accumulated masses of sediment, reinforced by entangled tree trunks and immersed debris. A complete occlusion of the outlet may occur. The reservoir must, however, be emptied occasionally; therefore, at the planning stage, facilities should be designed to allow floating objects to be evacuated through gates of approximate dimensions at the right places. The conduits of the bottom outlets must be straight and of sufficient size, controlled by gates which allow free passage of the flow.

It is recommended that trees in the reservoir area should be cut before it is filled.

1.2.2 Influence on Gates and Valves

At high flow velocity through the bottom outlets ($> 40\ m.s^{-1}$), cavitation and flow-induced vibration may be accompanied by the abrasive action of sediment particles, affecting the structures, linings, concrete surfaces, and particularly the apron, especially if the sediment is composed of hard minerals.

The emanations of gas - particularly of hydrogen sulphide - discharged by the dislocated and decompressed mud, may corrode the metallic parts of the gates and the equipment located in the gate chamber.

1.2.3 Influences on the Dam

The pressures exercised by the sediment deposits should be considered in the structural design of the dam.

Moreover, it is important to make sure that the designed characteristics of the structure are not altered by deterioration of the concrete. The chemical reactions within the deposits, particularly those pertaining to the sulphur cycle, as well as the degree of corrosivity of the water, must be taken into account. The concrete and concreting techniques, as well as the lining of the surface of the dam, should be foreseen by the design.

1.2.4 Influences on the Supervision of the Dam

Silt deposits may hinder the supervision of the dam. They can prevent the use of underwater means of supervision, such as submersibles, divers, television, etc. They hamper visual observation after having emptied the reservoir, because the surface of the dam is soiled by mud and access from the side of the reservoir is made impossible.

1.3 ECONOMIC AND SOCIAL IMPACTS

1.3.1 Consequences of the Reduction of Storage

The impact of reservoir sedimentation on the social and economic objectives of the project depends upon the size and the characteristics of the deposits. The consequences are very complex, because the dam usually serves multiple objectives, which may evolve over the years.

Sedimentation affects the storage capacity, which is the main asset of the reservoir. The loss of storage is particularly felt in connection with energy production, water supply for domestic use, industry and agriculture, and in discharge control. Sedimentation also affects the surface area of the reservoir, by reducing water depth and favoring the development of aquatic growth. This affects, sometimes, adversely, the use of the reservoir for recreation (fishing and hunting), fire fighting and public health.

(A) Influence on Energy Production

Hydroelectric plants are used to provide the peak demands for energy because of the flexibility of their operation. The reservoir is, therefore, an energy accumulator - the larger its capacity the more efficient it is. Hence, any reduction of the capacity will reduce the energy output of the associated power plant and the maximum power available during the most critical period of consumption, when the water input is insufficient.

(B) Influence on Agriculture and Industry

If there are alternatives to the production of water power, there is no substitute for water for irrigation. Although the most efficient techniques of irrigation may save water, they cannot do without it. Therefore, sedimentation of a reservoir serving agricultural purposes is far more critical than of that serving power production.

The effects of a well-designed irrigation project may often be spectacular with regards to agricultural production; in certain climates, irrigation is a necessary condition of life itself. The loss of storage may, thus, lead to shortages of water with catastrophic consequences, since they can cause the loss of the crops which are the result of a year's labor.

The deepening of the riverbed downstream from the dam as a consequence of trapping of sediments in the reservoir, may lay bare the intakes of water supply systems which need to be reconstructed in order to serve their objectives.

Industry is a large consumer of water, the water being used in the industrial process itself or for cooling. Water shortages may be very costly for industry, although the consequences are more transient than in agriculture.

Sediments ejected into the downstream reach of the river by floods, flushing, reservoir emptying, etc., may cause different nuisances: silt deposition in water supply conduits, obstruction of sprinklers in irrigation systems, clogging of heat exchangers in factories, disturbing the operation of wastewater treatment plants, producing spots on fruit, and thus reducing their commercial value, etc.

(C) Influence on Discharge Regulation

Reduction of the storage capacity also affects the capability to regulate discharges of supplemental water in time of low river discharges for environmental protection and for industrial or agricultural activities.

In the case of reservoirs for flood control, the loss of storage may have even more serious consequences because the flood plain dwellers downstream from the dam, believing that they are protected by the dam, usually neglect other measures of protection against possible flood damage.

1.3.2 Influence of the Reduction of the Water Surface

The reduction of water surface is caused by both the emergence of deposits and the growth of weeds in the shallows.

(A) Influence of Boating, Sailing and Other Outdoor Sports

Sediment deposition and growth of weeds can obstruct the access to marinas and hinder the movement of craft.

Mud is a serious nuisance to beaches which are otherwise suitable for swimming and water sports.

On the other hand, aquatic vegetation is an excellent shelter for birds and other aquatic fauna; it is thus welcomed by fishermen and hunters.

(B) Influence on Fire Fighting

Large reservoirs are well suited for refilling of seaplanes, used in some countries for fire fighting. A substantial reduction of the free surface can impede the action of the aircraft.

(C) Influence on Public Health

The same vegetation on the banks of the reservoir, which provides shelter for animals, also favors the proliferation of insects, whose unpleasant bite alone can discourage human presence in the area. More serious is the danger from insects which carry diseases such as malaria, onchocerosis, yellow fever, etc.

Chapter 2
Physical Processes of Reservoir Sedimentation

2.1 INTRODUCTION

In geomorphological terms reservoirs for water detention or retention are developed in two situations, either based on pre-existing lakes, or as entirely fresh impoundments on the floors of river valleys. Each has a morphology determined by its former state; and the basin shape, water circulation, and sediment supply strongly control the nature of accretion within the basin, and the speed of infilling.

According to Hutchinson (1957) eleven major processes contribute to the formation of natural lakes, from floor excavation to the blockage of the drainage by obstructions such as landslides. Of the 76 lake types recognized, few are suitable for artificial enlargement, as many are ephemeral, highly unstable, or collect little influent water (e.g., salt lakes, ice-dammed lakes, crater lakes). Principally, lakes originating from the action of rivers, or glaciers, near shorelines, or formed from substantial landslides, satisfy the requirements for reservoir construction.

A reservoir developed through the raising of a natural or pre-existing artificial dam at the exit of a lake inherits the water circulation, sediment supply and morphology of its predecessor, modified by the physical effects stemming from the impact of the newly flooded marginal areas.

River systems are perceived as evolving through predictable sequences of change. Narrow, winding, high-gradient valleys which typify the earliest phases, are succeeded by more open, gently sloping valleys with alluvial terraces along flood plains. In their extremes the valleys become very flat and the relief is severely reduced. Such a progression may take many millions of years to develop and within a basin the more mature features first appear in the lower reaches, whereas the high relief of youth is preserved for a long time in the headwaters.

The evolving valley, therefore, presents changing possibilities of sites for reservoir creation, in terms of both location and time. Deep, narrow, winding reservoirs typify headwater areas and 'young' terrain; whereas shallow, areally extensive reservoirs occur in lower land characteristic of more 'mature' sites.

The damming of rivers or the raising of exit thresholds to pre-existing lakes leads to the submergence of an established quasi-stable land surface beneath the impounded waters. The flooded terrain has features which initially exert controls upon the water circulation and sediment distribution pattern within the drowned area, but these controls decrease in significance as sedimentation advances. The impounded waters provide temporary base levels to the influent streams, which deposit solids in the relatively still waters, and also suffer from backwater effects leading to siltation of channels above the reservoir, broadening of areas liable to flooding, and deterioration in the navigability of the waterway. Below the dam the effluent waters are relatively depleted in suspensates, and unless there is controlled release of sediment-charged waters, initial degradation of the river bed is likely. Fine particles are winnowed from the channel bed until a stable, armoured floor is established and the channel achieves a newly adjusted gradient.

The two distinct origins provide basins of two fundamentally different forms. In the 'flooded valley' reservoir the deepest part of the basin slopes continually towards the dam and the dead storage area is defined by the dam configuration and the exit sluices. In the 'expanded lake' reservior part of the natural floor slopes away from the dam, and the dead storage area may include substantial deep basins physically separated from the dam.

Although both basin forms respond to the principal physical processes acting on the water column, such as wind-wave generation of circulation, thermal stratification, development of seiches, and the reception of sediment plumes from influent rivers (Sly, 1978); the distribution patterns of highly turbid inflows may differ considerably. In the 'flooded valley' reservoir high density flows may sweep the floor of the entire basin, to give deposits against the back wall of the dam (Gould, 1960, Lara and Sanders, 1970); in the 'expanded lake' reservoir such flows may enter the principal or subsidiary basins and their loads are retained in the dead storage space in areas remote from the dam.

2.2 NATURE OF THE SEDIMENT SUPPLY

The water discharge to a reservoir is largely determined by climatic controls, but factors such as relief, vegetation cover, agricultural activity, and the nature of the underlying rocks are also important. The discharge rises and falls with precipitation or snowmelt in the catchment, showing peaks and troughs which may follow each other within hours or days, but within a general seasonal pattern of variation, whether in temperate, tropical or polar regions.

Whereas pebbles and coarse sands roll or slide along the bed or bounce into the flow in saltation, the finer sands, silts and clays are carried throughout the water column, buoyed up by the natural turbulence of the water. In strong flood flows coarse bed load material is mobile, but in lesser currents the largest particles remain unmoved as only finer material migrates along the bed. Below a local threshold value no material travels as bed load and solids are transported only in suspension.

The estimation of the load carried by water has long provided a challenge to hydraulic engineers, who have produced a bewildering variety of procedures and formulae to facilitate load estimation (Graf, 1971; Yalin, 1972; Bogardi, 1974; Vanoni, 1975). Variables which have been linked with sediment discharge include bed slope, shear stress, fluid discharge, velocity, intensity of turbulence, fluid temperature, particle size, and bed configuration. There is no simple universally applicable formula which interlinks all these variables. Nevertheless, according to each approach, as the discharge past a point decreases, so the carrying power of the water falls, and as the competence to transport the load decreases, sediments become deposited, giving rise to bedforms which present higher resistance to flow and further reduce current speed and transporting potential.

Fine silts and clays of the wash-load are maintained within the water mass as it flows towards the reservoir. The fine particles have low settling velocities and are kept in suspension by the turbulent motion. Whereas the coarser particles move only while currents or bed shear exceed local threshold values, the very fine sediment moves whenever the water is in motion. Suspension concentrations vary through discharge events. The actual concentration levels encountered depend upon the rainfall intensity and the susceptibility of the soil material for entrainment. In dry terrain long intervals between storms enable mudflows to develop, but hyperconcentrations (>200g/ℓ) also develop in moister terrain and in the Yellow River basin mudflows containing 1600 g/ℓ have been recorded. In general, the more flashy the stream the higher the suspended concentration carried.

Peaks of flow and turbidity are coincident in semi-arid upland areas, but are often out of phase in more humid climates. In some catchments the turbidity peak preceeds the discharge peak in early winter (Miller and Piest, 1960) but undergoes progressive delay so that it lags behind maximal flows in late spring and early summer (Walling and Gregory 1970, Culbertson et al. 1972).

While the water peak travels through a basin with the speed of a wave motion, the suspension peak moves with the slower current flow so that the relative arrival times of the two maxima change through any one catchment (Figure 2.1). Thus, the greatest suspension loads may enter any one reservoir sometimes before, sometimes in step with, and sometimes after the discharge peak. In reservoirs low in catchments, suspension peaks normally follow flood discharge peaks, but suspension peaks normally arrive first in reservoirs high in the catchment.

The caliber of both the wash load and the bed material load particles transported by a river varies with season, as the particles respond to the shear stresses exerted by the flowing waters. During high discharges more sand is lifted from the bed than during low flows (Herb, 1980) (Figure 2.2).

Material in solution travels directly to the reservoir with the water as solutes and are dependent neither on current speed nor turbulence. The greatest loads are carried during peak discharges, although solute concentration levels are greatest during low flows The solute concentrations are very climate dependent, being higher in dry terrains, but often exceeding 50 mg/ℓ in temperate climates. In most rivers solute concentrations increase seawards (Langbein and Dawdy, 1964; Loughran, 1975).

Sediment transport problems are not uniform throughout the world, for erosion rates are dependent upon the combined effects of relief, climate and vegetation, but human activity may also contribute. As a result sedimentation problems in reservoirs and lakes vary greatly from continent to continent.

Whilst Holeman (1968) suggested that the rivers of Asia carried about 80% of the sediments entering the world oceans, on the basis of precipitation and relief Fournier (1960) predicted greatest sediment yields from the seasonally humid tropics with less in the equatorial tropics and arid zones. However, very different suggestions were made by Fleming (1969) who analysed sediment yield according to vegetation zones and demonstrated maximal yields in desert and scrubland and lowest yields in the mixed broadleaf-coniferous vegetation belt. Thus reservoirs established in arid or semi-arid areas are most likely to suffer from severe sedimentation problems. Those in temperate zones are less likely to experience such difficulties.

Although in most reservoirs sedimentation is principally from inorganic material, in many shallow lakes, in areas of low relief and where rivers carry small loads the indigenous phytoplankton populations may contribute important quantities of particulate matter to the bed. Phytoplankton populations rise and fall with the season, different species 'blooming' at different times of the year. In some lakes and shallow reservoirs the productivity defined in terms of organic carbon generated is considerable. Biogenic silica from the skeletons of diatoms is also an important component of the system. Locally the organic contributions are very important and should not be overlooked. However, problems of assessment of these contributions will not be further examined in this report.

2.3 MORPHOLOGICAL EFFECTS OF RIVER IMPOUNDMENT

In a stable river bed, erosion, sediment transportation and deposition are in equilibrium. Interference with that condition by the erection of a dam disrupts the balanced state through raising the local base level of the streams above the reservoir. The immediate response of the system to this aberration is to deposit material within the receptor basin in an attempt to reestablish a uniformly graded river profile. Similarly, the river responds to changed conditions below the dam. The nature of the responses may be anticipated by reference to the general principles outlined by Lane (1954), who suggested that a balance exists between the sediment load and the particle size on the one hand and the dynamics, represented by the stream slope and discharge on the other (Figure 2.3). In the long term the reservoir will approach a largely silted condition, although suitable management may delay the complete filling. In engineering time substantial changes of storage capacity, aggradation of the river course above the reservoir and scouring of the bed downstream of the dam are problems directly associated with the reservoir construction.

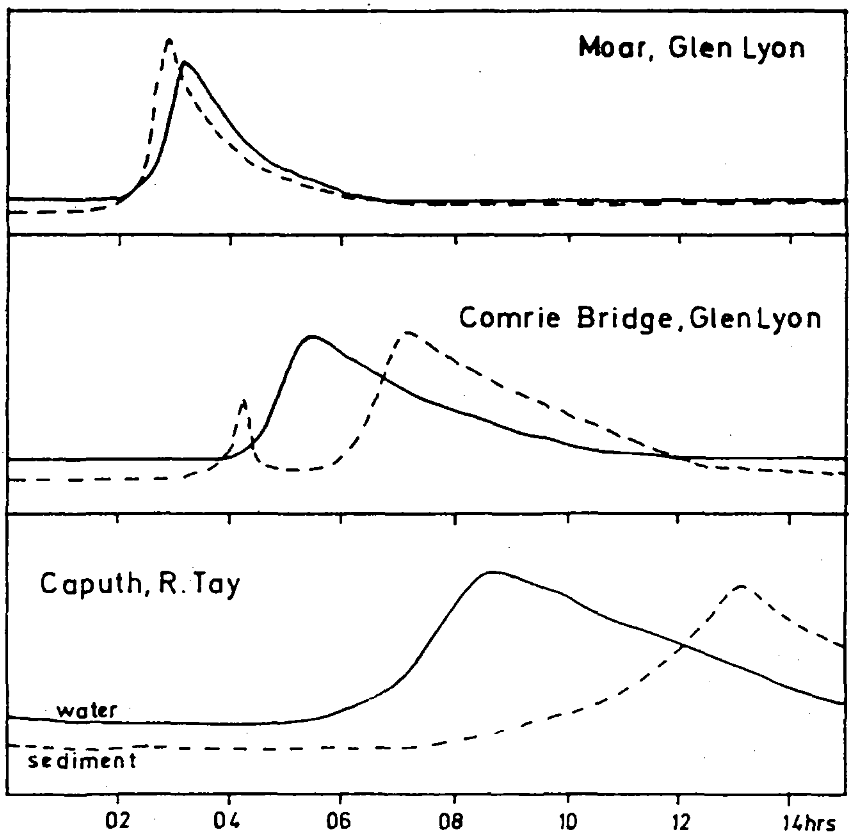


Figure 2.1. Variation in arrival times of peak water discharge (solid line) and peak suspension concentration (broken line) at three stations in the Tay drainage basin, Scotland

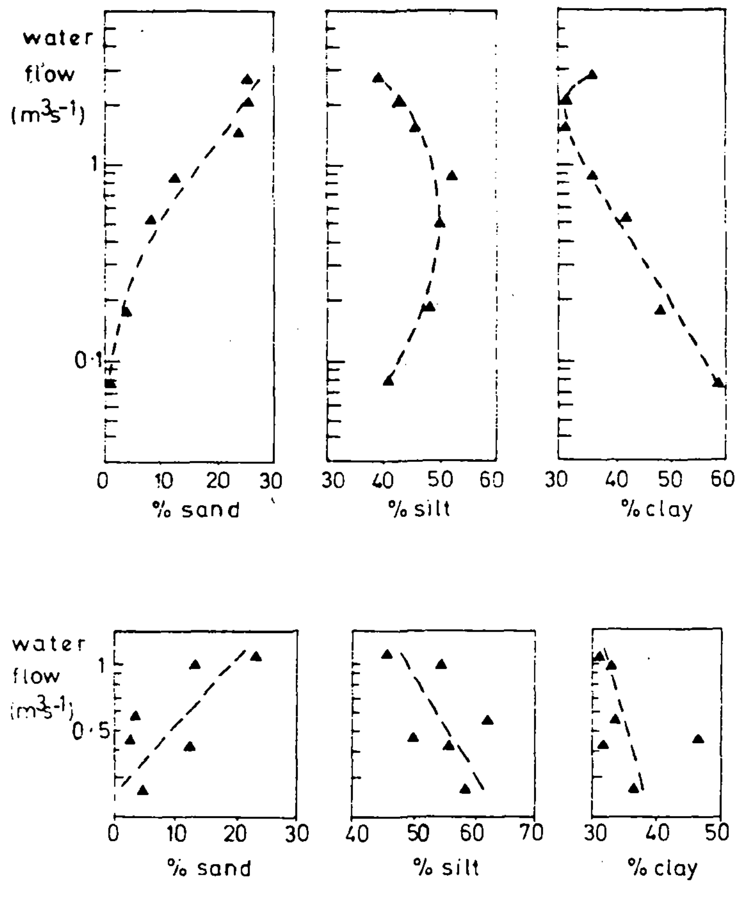
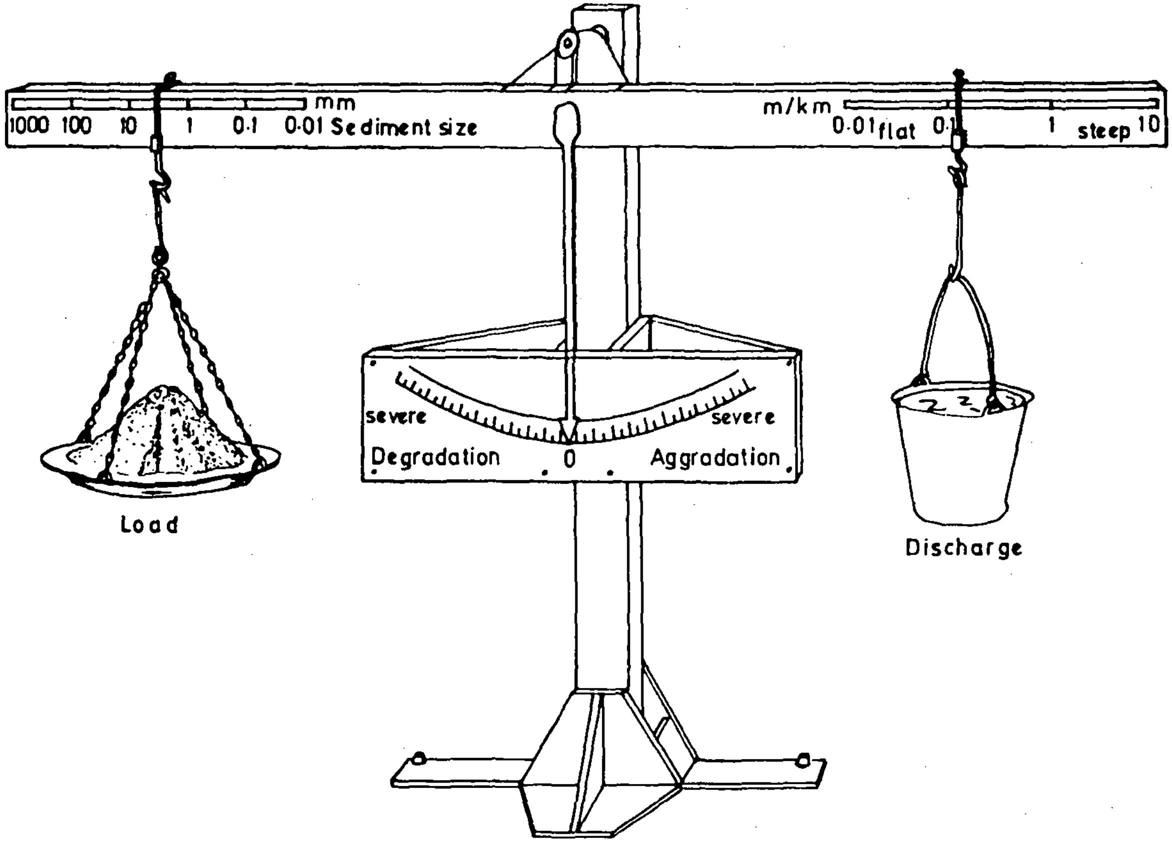


Figure 2.2. Relationships of sand, silts and clay content of suspended sediment to water discharge. Upper plots from Manor Run, Norbeck, 1967-72. Lower plots North Branch Rock Creek, Norbeck, 1969-75. (After Herb (1980).



SEDIMENT LOAD X SEDIMENT SIZE $\propto$ STREAM SLOPE X DISCHARGE

Figure 2.3. *Interrelationships between factors contributing to establishing a stable balance in a river channel. (After E.W. Lane, 1955).*

2.3.1 Aggradation Above the Reservoir

The water in a reservoir moves relatively slowly and inhibits the forward motion of the waters in the influent streams. Backing up of the waters takes place, and the associated reduction of flow is accompanied by a decrease in stream competency to maintain sediment motion. The level to which backing up occurs is dependent partly on the water level in the reservoir, as this defines the local energy base, and partly upon the water discharge.

When the water level is low the influent stream accelerates as it enters the basin and the increased currents scour the channel floor, eroding the sediments and carrying them further into the retention basin before redepositing them. High river discharges coupled with low-standing water levels may give rise to substantial entrance channel scouring.

Low flows are associated with little backing up but in floods the backup may extend for several kilometers above the reservoir. Deposition of bed load in the reach of decelerating currents leads to a rise in channel floor level. This increases the possibility of flooding in the low lying ground bordering the thalweg (sic). Agricultural land in the flood plain may be lost through increased frequency of submergence by flood waters, and also as a result of saturation associated with raised watertable levels. Deposition of sediment brings fresh material to the flood plains in 'warping'.

Shoaling of the channel bed through sediment deposition decreases the water depths and presents increasing and often highly mobile hazards to navigation. Areas of shallow water become infilled first in a progressive change in the regime of the influent stream.

Although sediment is deposited above the reservoir, much material is carried into the standing body of water, where rapid flow deceleration occurs. The coarsest material is dropped almost immediately while progressively finer particles are carried further into the basin. The resultant deposit of sediment builds out into the reservoir around the river mouth, forming a delta whose surface may extend to the highest water levels. The exposed surface of the deltas may become covered by reeds, scrub or low trees. Where levee lined channels build out into the reservoir, quiet backwater areas are established in which weed growth effectively prevents water circulation.

The delta top may be of boulders, pebbles, or coarse sands according to the nature of the transported loads. Concentration of coarse material may occur through sediment reworking by waves during high water. Delta forms vary greatly but have braided patterns in coarse sediments (Worsley and Dennison, 1973). Where wave activity is significant the distributary mouth spits and bars are reworked and the delta assumes a smooth plan.

Tributary streams entering on the flanks of reservoirs also create deltas, which not only restrict circulation patterns, but may ultimately build across the water body in areas of rapid siltation.

The deposited sediments accumulate to produce delta slopes at angles approaching the angle of repose of the sediments, so that steep inshore slopes become progressively gentler offshore.

The rates of deltaic advance into lakes and reservoirs have been estimated for Lake Mead - 3000 m average 1939-1948 (Sundborg, 1964), the Terek delta on the Caspian advanced 300 m a year (Holmes, 1955) and the Lillooet River of

British Columbia builds forward at a yearly average of 10 m (Gilbert, 1975). The rate of progression is dependent not only on the river discharge and catchment characteristics, but also upon the geometry of the receptor basin.

2.3.2 Degradation Below the Reservoir

The relatively static body of water retained in the reservoir provides conditions ideally suited to the settling of sediment of all sizes. Suspension concentrations are severely reduced, and bed load is probably completely eliminated, so that any water permitted to overspill or released from the sluices is relatively clear. The release of turbulent, sediment-free flows which have an unsatiated potential for carrying sediment encourages entrainment of suitably sized particles from the river bed if they are available. Since most readily eroded particles are scoured from the bed first, it becomes depleted in fine particles but relatively enriched in coarse material. The resultant stable bed is coarser and has become armored. As the upper reaches become armored the active reworking of the bed progresses downstream, so that armoring also migrates with time. The process of armoring, widely recognized in fluvial hydraulics (Gessler, 1965), has been examined in the context of reservoir outflows on the Nile (Hammad, 1972). A rippled, armored, bed surface, as recorded on the Colorado below the Hoover, Parker and Imperial Dams (Brooks, 1958), was predicted as the stable end product of degradation below the Asswan High Dam. Hammad suggested that the armored surface usually develops before appreciable bed slope change occurs if suitable materials are present.

Whether gravel beds are stable depends principally on the stream competency downstream of the dam, but in general, scour and slope changes have limited impact in gravel bed streams, but they are much more important in sandy water courses where downcutting and regrading may be lengthy processes.

When periodic sudden releases of sediment-charged waters occur in attempts to maintain the reservoir storage capacity, the discharged flows have higher relative specific gravity than the normal effluent waters leaving the reservoir and give rise to additional scouring and associated slope changes. Thus, where a self-scouring management programme is followed, the bed stability of the stream below the dam is not so readily established. The water course requires careful monitoring and the possible installation of additional protection measures to ensure that the newly degraded slopes do not undermine the toe of the dam through headward erosion.

2.4 MOVEMENT OF SEDIMENTS IN RESERVOIRS

The behavior of sediments within reservoirs and lakes is determined by the various forms of water circulation, which also select areas of scour and accretion. The dynamic forces and mechanisms involved are many and varied, and several processes may interact, so the study of sedimentation in reservoirs demands the examination of many phenomena.

Three natural sources of energy controlling the physical processes are active in the waters, viz. the effects of wind, of river flow and of solar heating (Table 2.1).

Table 2.1 Physical processes affecting sediment movement in reservoirs and small lakes (after Sly, 1978).

Natural Agency	Wind	Solar Heating	River Input
Controls	orientation size, depth shape of basin local topography duration Coriolis force	latitude altitude local topography water depth	water discharge sediment concentration lake shape water temperature Coriolis force
Responses	wave generation water currents upwelling coastal jets seiches	stratification mixing overturn ice cover internal waves	turbid plumes density flows delta growth

The activities of each source providing the energy driving the system are determined by the local climate, and their impacts vary in relative importance from site-to-site. Tidal motion and differential pressure seiches occur only in water bodies much larger than most reservoirs.

2.4.1 Wind Effects

When wind blows across a reservoir it exerts a shear stress on the water surface, and the resulting momentum transfer causes the surface water to move in the direction of the wind. Waves, water surface currents and associated counter-currents, and Langmuir circulations become established.

Wind-generated waves follow oscillatory motion in which the height (H), wave length (λ) or crestal spacing, and speed of motion (C) are well defined. These characteristics result from the wind energy expended and this is linked to speed, duration and fetch (the unimpeded distance across which the wind blows). The greater each factor, the greater the energy input and the larger the waves. The depth of water in the basin also exerts an influence. In some reservoirs water depths are sufficient for deep water waves to form in which

$$C^2 = \frac{g\lambda}{2\pi} \tanh \frac{2\pi d}{\lambda}$$

where C is the wave velocity, λ the wavelength and d the water depth. The water particles follow orbital paths in which the circumferential velocity $\pi H/\lambda$ may reach over a meter per second. As they move into shallow water the deep water waves become solitary so that once the depth falls to half the wavelength, then $\tanh 2\pi d/\lambda$ approaches $2\pi d/\lambda$, in which case

$$C^2 = gd.$$

Theoretical curves for the generation of waves in deep and shallow waters (Figures 2.4 and 2.5) have been given by Bretschneider (1965) and The Coastal Engineering Research Center (1978).

When the waves run into shallow water their horizontal motion components are maintained while the vertical components decrease until ultimately the deformed circulation paths cease to close and mass movement of the water takes place. The

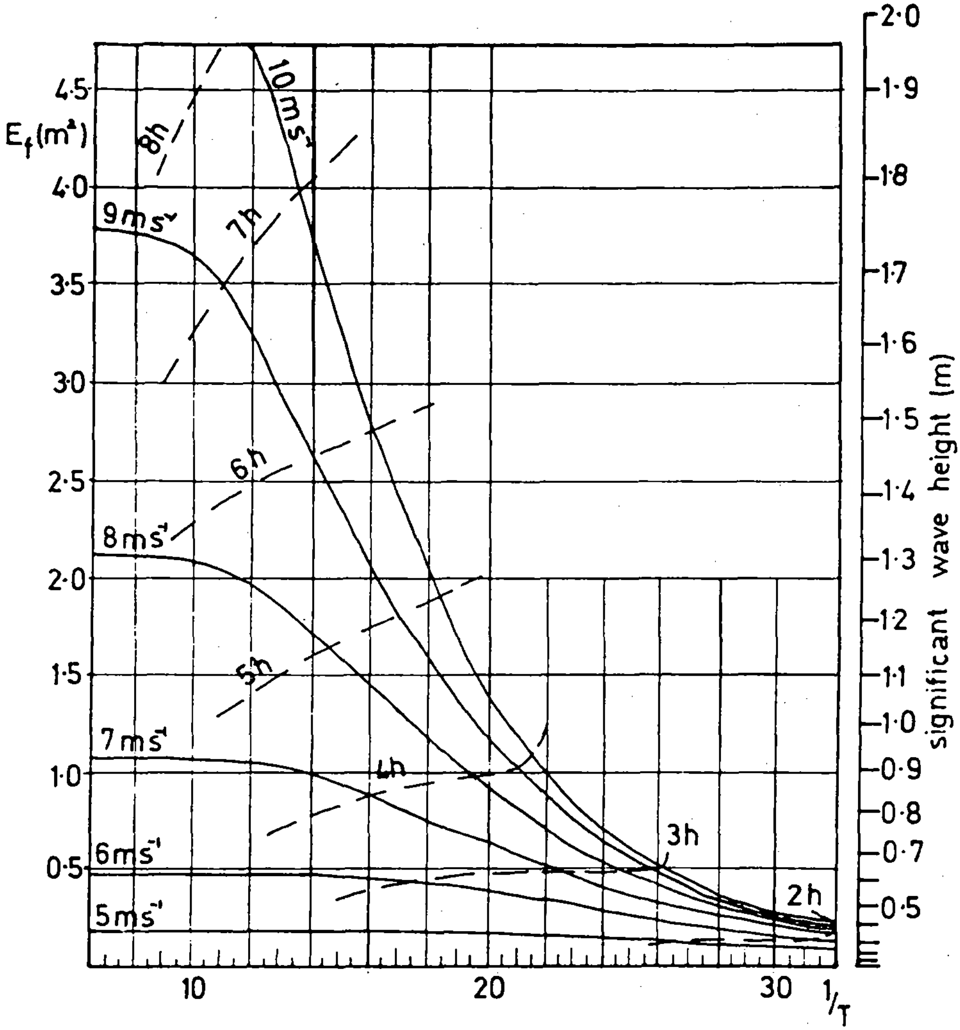


Figure 2.4. Co-cumulative spectra curves for wave generation. E_f values are related to wave energy and determine the height characteristics of the composite wave motion. Continuous lines signify wind speed (after C.E.R.C., 1966). C.E.R.C., 1966. Shore Protection Planning and Design Tech. Report No. 4.

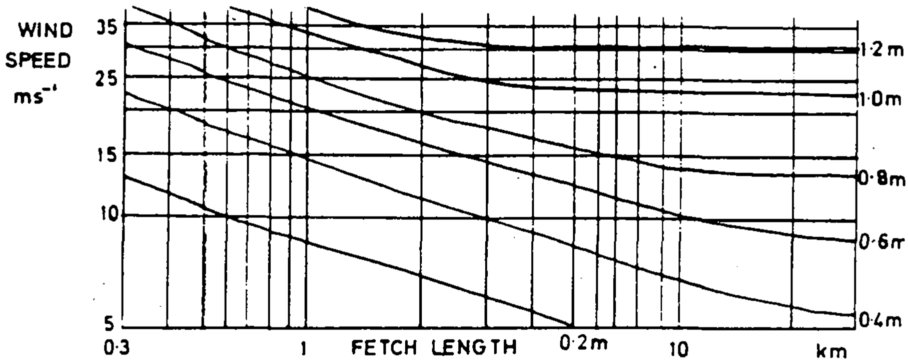


Figure 2.5a. Wave prediction diagrams for water of constant depth of 3 m

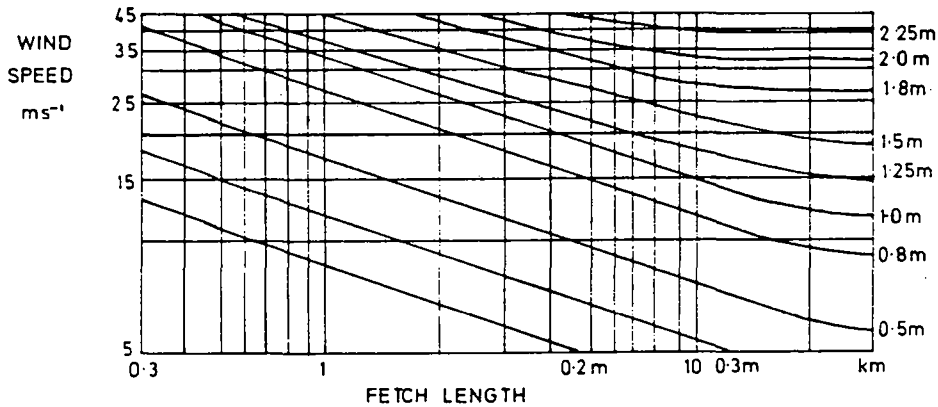


Figure 2.5b. Wave prediction diagrams for water of constant depth of 6 m

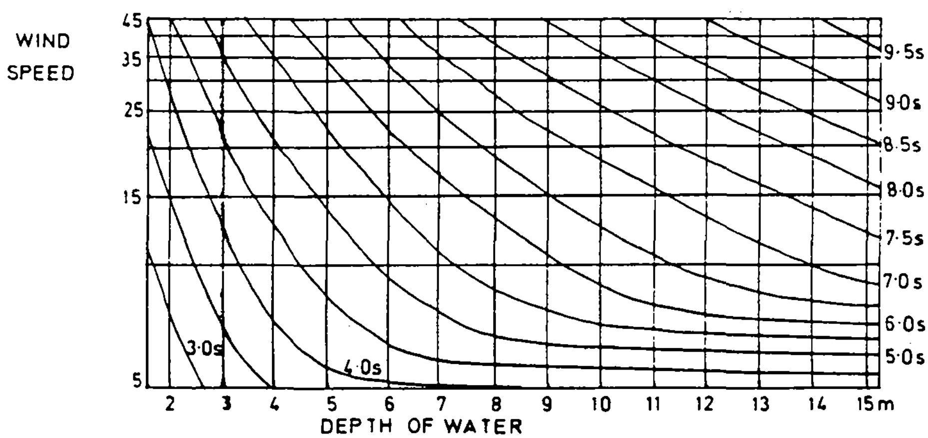


Figure 2.5c. Forecasting curves for significant wave periods

spilling crest of the breaking wave runs forward to dissipate its energy on the shore, entraining sediment as it travels and carrying it alternately shorewards and basinwards as swash and backwash. The onshore energy flux has not been successfully determined, although Longuet-Higgins (1952) estimated the mean wave energy per unit area of water surface to be $1/8\ \rho gH^2$ for the root mean square of wave height.

The oscillatory currents of passing waves ensure that deposited sediments are frequently reworked in waters shallower than half the wave length. While fine sediment may be resuspended, sands remain to form ripples on the bed.

When waves travel across water of varying depth those parts in deep water travel faster than those in shallow water, and the wave crest line bends. Such wave refraction towards headlands occurs in the ideal situation, and refracted waves sweep round the coasts of offshore islands. Similarly, waves passing along a reservoir are slowed toward the shore, so that the wave energy is not expended directly onto the shore, but rather, obliquely to the shoreline. Beach face particles, therefore, move obliquely up the face with the swash but roll down slope under gravity so that they progress along an irregular almost cycloidal path. Longshore currents generated by wave refraction are dependent upon wave height, period, angle of approach and the beach slope; and they transport suspensions along the shoreface, but in the waters offshore. The two modes of beach sediment transport ensure that small inlets entrap the sands and fine gravels (Hands, 1970; Rukavina, 1970; Hakanson, 1977, Csanady, 1978). The finer materials may move offshore into the basin itself to settle as 'pelagic' sediment (Fricbergs, 1970).

Much of the breaker energy is directed at the shorelines of the reservoir and erosion of unconsolidated bank material results, with the production of shoreline cliffs. The fine particles are taken into suspension and carried away from the shore, while the coarser particles remain to form a beach berm and slope system close to the water surface level. The outer margin of the beach slope becomes steep, and successions of steps are present where beaches have developed during falling water levels. In general, static water levels are associated with broad shoreline surfaces.

Scouring is not restricted to the beaches as the vertical component of the waves lifts small particles from the bed at any stage during wave shoaling or passage of the wave over shallow areas. Winnowing of the marginal areas through wave entrainment creates marginal zones of coarse sediment and deeper areas dominated by fine particles. The behavior of lake margin beaches is poorly understood and has received little attention outside the Great Lakes of North America (Fox and Davis, 1970, 1976).

The shear stresses exerted by the wind also induce forward motion of the surficial waters of the basin. The mobility of the skin waters is often high, and currents of 1-3% of the wind speed are typically encountered (Bengtsson, 1978; Haines and Bryson, 1961). The surficial current drift may transport considerable volumes of water along the reservoir, and at depth counter currents are established returning water to the upwind end of the basin. In shallow lakes the return currents develop near the lake shores. These basin floor currents flow at very much lower speeds than the surface drift, generally an order of magnitude less (3 cms^{-1} against 30 cms^{-1}). Such currents may be sufficient to carry the concentrated nearbed settling suspensions to the windward end of the basin where enhanced deposition may occur.

The surficial currents, like all moving fluids, are subjected to geostrophic deflecting forces of the *Coriolis effect*, so that in the northern hemisphere currents are deflected to their right and their left in the southern hemisphere. Current deflections of 20° from the wind direction are common. In Lake Windermere, England, George (1981) demonstrated 4°-38° deflections, the greatest deviations occurring at low wind speeds. The currents experience greater deflection with depth below the surface to produce the 'Ekman spiral' effect (Rossby and Montgomery, 1935). Rotation of current direction with depth is recorded from Lake Mendota, U.S.A. (Shulman and Bryson, 1961) from Lakes Velen and Möckeln, Sweden (Bengtsson, 1978) and from the Loch Earn reservoir, Scotland (Duck, 1982). The angular deviations of current from wind direction mean that flows may be onshore or offshore although the wind is shore-parallel. In reservoirs, such as the east-west trending Loch Earn, net movement of suspensions away from the northern shore and towards the southern shore are to be expected from the prevailing westerly winds.

The motion of a wind over standing water often forms wind-parallel horizontal axis helices with clockwise and anticlockwise rotation in alternation, manifest as a streaked appearance to the water surface. The associated currents sweep towards convergence zones, reaching to 30% of the wind-generated surface current speed (9 cms^{-1} with 30 cms^{-1}) (Ottesen Hansen, 1978). These Langmuir circulations are confined to the epilimnion in stratified waters, but may extend to the basin floor in isothermal conditions. Minor linear sediment concentration bands result from the sweeping effect of the helical systems.

Prolonged wind activity along a reservoir produces a surface gradient in which water level is raised in the downwind sector. Sudden cessation of the wind releases the water, and an oscillation is set up, whose period and height depend upon the geometry of the basin. Similar oscillations follow rapid changes of barometric pressure, and both form seiches, which resonate through the basin, generating weak currents capable of transporting suspended sediments. All reservoirs resonate. Each has a characteristic number of nodes and axes of oscillation which are geometrically defined, so that seiche-induced currents (1-4 cms^{-1}) are often complex. Some may carry small quantities of sediments to recognizable sites, but in most reservoirs these patterns are over-ridden by the effects of other factors.

2.4.2 Solar Heating

A climatic effect observed most clearly in lakes and reservoirs of temperate regions results from the thermal input to the water surface.

In spring the waters are virtually isothermal with temperatures a little above 4°C from the bed to the surface. As summer advances the surficial waters become heated by solar radiation and although wind circulation ensures some downward heat transfer the upper waters become steadily more buoyant as their density decreases with increased temperature. Ultimately, downward mixing and heat transmission cease and an upper zone of relatively warm, low-density water, the epilimnion, rests above a zone of colder, denser water, the hypolimnion at depth. Between these water masses, which establish independent patterns of circulation, is a pycnocline, or density gradient, normally observed as a thermocline, where temperature gradients may exceed 1° C/m (Kindle, 1927).

In late summer heat loss from the well-mixed epilimion waters reduces temperatures to those of the hypolimnion. The thermocline ceases to exist, density differences disappear, and the two water masses function as a single circulating mass again. Continued cooling reduces surface temperatures to 4°C, in which state the waters are most dense; they sink and there is an interchange of bed and surface waters. This 'overturn' carries nearbottom waters to the surface charged with high suspension loads, giving turbid waters and providing clarification problems if the reservoir is used for public water supply.

A reversed thermocline develops with cold waters at the surface and the 4°C water at depth as stratification reappears. As the density differences between epilimnion and hypolimnion are slight, strong wind induced circulation may lead to a breakdown of the layering, which becomes reestablished in calm conditions (Ragotzkie, 1978). Although the surface waters may freeze, the thermal structure remains stable until spring warming raises the epilimnion temperatures to 4°C at which stage free interchange of the waters is reestablished and the thermal stratification cycle recommences.

While temperate reservoirs experience twice yearly mixing periods (dimictic), those in high latitudes or altitudes may not warm above 4°C and show only one phase of mixing annually (monomictic). In low latitudes water temperatures never cool to 4°C and they may also show only one annual phase of mixing. Deep tropical lakes exhibit permanent stratification in which the pycnocline is controlled by variations in the solute content of the waters (Yuretich, 1979).

The pycnocline provides a physical surface between two independently circulating fluids and both waves and seiches may develop along this interface. The internal seiches may generate swifter currents than the free surface seiches and suspended sediment may be transported along the pycnocline by these phenomena.

The thermal regime, therefore, controls the motion of the water masses, the oxygenation of the water, the chemical environments in which the organisms live and the pelagic sediments settle.

2.4.3 River Input

When a river flows into a reservoir it carries its load in three forms, as bed material, in suspension or wash load, and in solution. The three loads vary in relative abundance and concentration through the year, as do the sizes of the particles in transport. The sediments carried in suspension, normally form 90-95% of the solids load (Parker *et al.*, 1964). They travel with the momentum of the river water and, in combination with the dissolved load and with water temperature, their concentration defines the density of the waters entering the reservoir.

The path followed by the sediment-laden inflowing river waters is determined by the density regime of the reservoir. In summer, when the river is stratified, the inflowing warm river waters may be of lower density than the epilimnion, in which case, they flow at the surface of the lake as an *overflow*. More commonly, the river water density lies between that of the epilimnion and the hypolimnion, in which case, the turbid waters flow along the interface at the thermocline in the form of an *interflow*. Inflows colder than the reservoir waters, or particularly heavily charged with sediment, have high density and travel into the lake basin along the floor to provide *underflows* or turbidity currents (Figure 2.6).

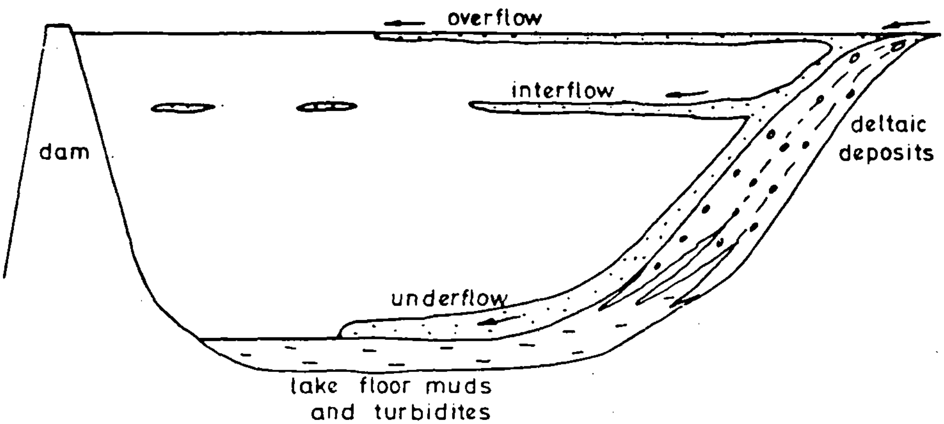


Figure 2.6. Distribution mechanisms and resulting sediment types proposed for clastic sedimentation in dipotrophic lakes with annual thermal stratification.

Both overflows and interflows enter reservoirs as linear features, but the wind-driven circulation ensures that the sediment-enriched waters become widely spread at those levels, the former being seen as discoloured plumes at the surface. The interflows or underflows exhibit abrupt colour changes round the plunge point where the masses sink below the surface waters.

The fine sediments may remain in suspension in the upper waters or are held at the pycnocline, where substantial concentrations may be held without significantly changing the density of the upper waters (Matthews, 1956; Sturm and Matter, 1978, p. 162, Table 2.2).

Table 2.2. Variations of density with water temperature and sediment concentration in Lake Brienz and the inflowing Aare River, during floods (after Sturm and Matter, 1978).

Water depth	Temperature °C	Suspended matter $mg\ell^{-1}$	Density water	Density suspension	Suspension density-water
River	5.4	21400	0.9999842	1.0128435	128593 x 10^{-6}
Lake Surface	13.8	10.7	0.9992987	0.9993051	64 x 10^{-6}
0.5 m	13.7	4.3	0.9993123	0.9993149	26 x 10^{-6}
10 m	11.0	40.0	0.9996328	0.9996568	240 x 10^{-6}
26 m	8.4	4.8	0.9998509	0.9998538	29 x 10^{-6}
110 m	5.6	54.7	0.9999795	1.0000123	328 x 10^{-6}

The sediments trapped at the thermocline (Nydegger, 1967) migrate with the circulation of the upper waters and are dispersed across the basin. Interflows in Lake Constance have been traced 20 km along the right bank of the lake (Auerbach, 1939) probably exhibiting the influence of the Coriolis deflection (Hamblin and Carmack, 1978; Wright and Nydegger, 1980). Preferential motion streams may concentrate interflow sediments into well-defined paths (Sturm, 1976).

When isothermal conditions are reestablished, the fine grained particles settle to mantle the entire basin floor. This well-defined layer of 'pelagic' sediments may form part of a laminated or 'varved' sequence (Østrem, 1975; Gustavson, 1975). In high latitudes and altitudes in winter, surficial ice restricts circulation from wind activity and the general reduction of currents permits settlement of suspensions, so that varved sequences have often been linked to circannual cycles of deposition, but their origin is not strictly limited to this pattern.

Where no thermocline develops homogeneous sediment sequences accumulate through continuous pelagic sedimentation, as in many tropical lakes (Ashley, 1975).

The occurrence of turbidity currents or underflows is strongly dependent on the nature of the transport from the influent rivers. Where rivers are highly charged with suspensions, as in the first flood of a season, the entering plume plunges down the delta slope towards the basin floor; (Gould, 1960 a, b; Lara and Sanders, 1970; Gilbert, 1975; Lambert *et al.* 1976; Zhang, 1976; Fan, 1981). Usually the underflow runs down clearly-defined pre-existing channels across the outer delta slope, overspilling from low marginal levees only in the lowermost sectors (Sturm and Matter, 1978). The occurrence of underflows is often linked to flood periods and may recur during one year, or may be confined to major run-off events such as 25-or 40-year floods, according to locality.

The density currents plunge below the clear reservoir waters when the suspension concentration rises above some density threshold. Effectively, this occurs when the densimetric Froude number falls to 0.6, a value which decreases as the load increases ($F_d = V^2/\frac{\Delta\rho}{\rho}gh$). In Lake Mead the density flows reaching the dam had densities of 1.006 to 1.02 (Gould, 1960); whereas, in the Sanmenxia Reservoir densities of 1.061 have been reported from turbidity flows.

In the Sanmenxia in the middle reaches of the Yellow River of China, Fan (1981 Figures 2.2 and 2.3) has shown the unsteady nature of the hydraulic characteristics of the density flow, such as velocity and depth of the current, as well as concentration and size of particles in suspension for flows lasting for periods of 8 hours (Figure 2.7). He stressed that during early siltation flows were confined to channel floors, but that once the channels had filled with sediment, the density currents were able to spread over broader fronts of advance although they moved more slowly at this stage.

The volumes of sediment transported by density flows vary from site to site, but in the Naodehai Reservoir on the Liuhe River of China, three successive unusually severe flood peaks in an eleven-day period in late July 1963 yielded a total of 79.1 x 10^6 T of material most of which accumulated in the basin as deposits from density flows (Fan, 1981). In such sites the entire depositional regime is dominated by turbidity flows, which may occur many times in one year. Elsewhere the flows are less frequent, but yield characteristically textured deposits. In the less extreme case of Lake Brienz one turbidite mapped over an area of 11 km^2 contains some 5 x $10^6 m^3$ of sediment (Sturm and Matter, 1978). In such sites the 50-year flood turbidity flow deposits interfinger with

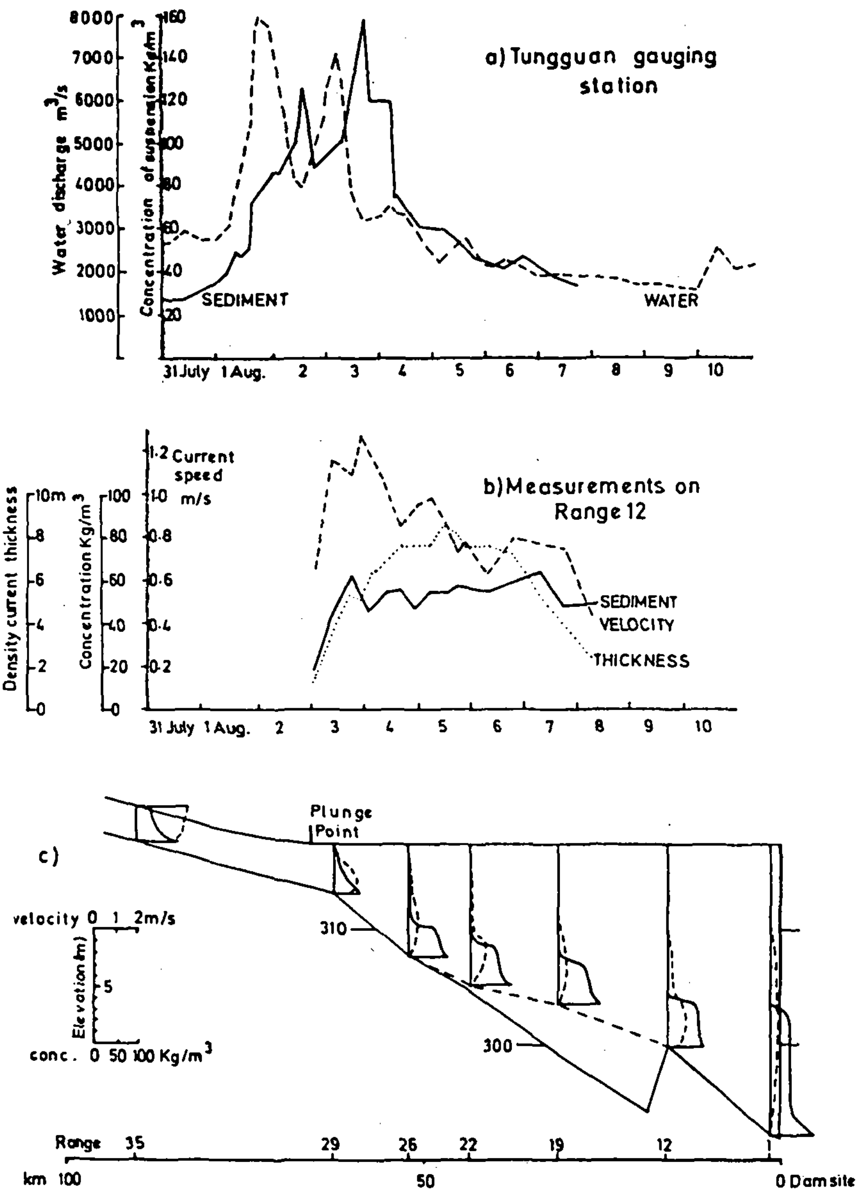


Figure 2.7. Unsteady density currents in Sanmenxia Reservoir (Fan, 1981) (a) Inflow hydrograph, (b) Hydraulic features of flow, (c) Density current. (Broken line velocity, solid line suspension concentration.)

with materials laid down under the influence of other processes.

Not all turbidity flows result from river input effects, for they may also be generated through subaqueous landslides from deltas or the margins of the reservoir basin. Such slips, which result from overloading of accumulations of weak sediment, but may also be seismically induced; occur independently of stratification in the water column.

Sediment particles dispersed in the water column settle towards the bed. They may be intercepted and held or retarded at the pycnocline, but when isothermal conditions prevail, they continue unimpeded to the bed at fall velocities, dependent upon their size and density and the viscosity of the water.

Large particles settle swiftly from suspension leaving behind the fine silts, clays and finely dispersed low-density organic particles. The fall velocities of clay particles are measured in a few tens of centimeters per day and they may be carried around the reservoir by the slowly circulating waters as they approach the bottom.

In most reservoirs fragments of minerals, such as quartz, feldspar, mica, and the clay minerals, dominate the inorganic part of the suspensions. All minerals are of regular crystal lattices, which have unsatiated ionic charges along fractured surfaces, so that many mineral faces possess -ve charge while the edges exhibit +ve charges. These charges encourage the suspended particles to come together to form ionically banded clusters, aggregates or flocs. The settling of flocs is swifter than that of individual particles, but is still relatively slow, for the aggregates form open structures with water between the component parts.

The probability (J) that particles will come together to form flocs is low in dilute suspensions, but increases as the number of particles present increases. It also rises with the size of the particles. The juxtaposing of many fine particles increased the probability of interparticle impacts, and gives use to perikinetic flocculation. However, still more collisions may be encouraged when velocity gradients are present to give orthokinetic flocculation, as may occur in shear across the summer pycnocline or near the bed with gentle currents. The probability is expressed in the form

$$J = n_i\, n_j\, r_{ij}\, \frac{dv}{dz}$$

in which n_i and n_j are the number of particles of radius, r_i and r_j and $\frac{dv}{dz}$ is the velocity gradient.

Einstein and Krone (1962) showed that in suspension concentrations of no more than 100 $mg\ell^{-1}$ with particles of 0.3 μm diameter flocculation in the presence of low shear, permitted the growth of flocs containing up to 600 primary particles and which had the same settling velocity as particles 2 μm in diameter.

Orthokinetic flocculation also results from the motion of slow residual counter currents from wind circulation in Langmuir cells, or in seiche node areas, all of which serve to bring dispersed particles together in shear and stimulate floc formation. Similar flocculation certainly occurs at the limits of density flows, although excessive shear during peak motion destroys the delicate structures.

There is some evidence that flocculation may be pH dependent, being stimulated by the more alkaline waters and discouraged in acidic systems (Döbereiner and McManus, 1982). Aggregation has been suggested as a mechanism whereby silts and clays entering the Highland Creek Reservoir of California are able to settle within the reservoir (Trujillo, 1982).

As the interparticulate bonding is largely ionic, the presence of salts in the waters may effectively decrease the thickness of the electrical double layer around the small particles and encourage aggrevation. Although a process more widely recognized in estuarine waters, it is also detected in reservoirs impounding brackish waters, as demonstrated in Lake Mead by Sherman (1953). The salt concentrations stimulating flocculation in this way may be very low, in the order of 10 $mg\ell^{-1}$ and these levels of salt concentration and far greater are present in reservoirs in most parts of the world. The solutes are not all of inorganic salts, but often contain organic fluids leached from the catchment soils or from decaying organisms within the reservoir waters. Organic fibres, diatoms and zooplankton, which may have highly charged surfaces, also join in the formation of aggregates in the water, (Holmes, 1968). The largest particles involved in floc making were about 24 μm in diameter, very similar to the maximum of 25 μm in seawater (Favejee, 1939). Once deposited the lightly held flocs may become re-entrained with the application of low shear stresses by currents; however, frequently the flocs break to form smaller units. Flocculent, homogeneous, fine deposits of low density, which produce 'ghost' returns above the more solid bed detected on echo-sounder traces and present little resistance to bodily penetration by divers, are recognized in many reservoirs. These deposits often contain substantial quantities of organic matter, 10-20% by weight (determined by H_2O_2 methods) or nearly 50% by volume in many Scottish basins. The importance of organisms and organic material depends largely on the nutrient input to the water from the catchment. Those deficient in plant nutrients are oligotrophic, those with moderate amounts mesotrophic and eutrophic waters contain abundant nutrients. The trophic condition determines the organic productivity of the waters and, thereby, the contribution made by phytoplankton to the bottom sediments. The organic materials serve to bind together the fine muds giving them a gelatinous appearance and imparting some strength to resist re-entrainment.

Both fine particles and organic detritus scavenge chemicals from the waters onto the particle surfaces. In some cases, this leads to enrichment of the sediment in potentially toxic substances so that entrainment of bottom sediments during the overturn period can be detrimental to the condition of the waters. Reducing conditions are present at shallow depth below the bed at many sites and the overturn may bring waters depleted in dissolved oxygen to the surface. Thus, the quality or condition of water in many dimictic lakes varies considerably through the year.

As the organic materials decay, gases such as methane are evolved. Although often freely released to the overlying waters, it is common for gas pockets to form below bed level. When this occurs below slopes, the gas pockets weaken the sediments and stimulate sliding and collapse of the sediment pile.

The pelagic sediments accumulating as loose aggregates on the slopes of the reservoir margins have little inherent strength. As the superincumbent load increases, a condition is reached in which the load exceeds the ability of the bed to support it. Failure may occur through floc-shearing and dewatering, through gas escape, or it may result from seismic vibrations. Whatever the cause, subaqueous mass slides, debris flows or slumps develop, with sediment moving downslope, sometimes as discrete blocks which preserve layering, but sometimes as structureless masses in which fragments of layered sediments are disoriented *in a chaotic mixture of water sediment. Distinct armchair-shaped* scoop slides resembling small rotational landslides are sometimes present. These structures, readily observed using sideways scanning sonars, may permit considerable masses of sediment to move towards the slope foot, and in suitable sites, the slides may impinge upon the dam, sluices and valve gear. During drawdown slides of this nature may be induced by removing the water from the reservoir sediments (Rodine and Johnson, 1976; Prior *et al.*, 1979).

2.5 GEOMORPHOLOGY OF LAKE FLOOR DEPOSITS

The many modes of formation and situations in which lakes may occur naturally, and the progression of topographic styles of river valleys which may be dammed, mean that there is an initial lack of morphological similarity in reservoir basins. Each inherits a structure generated during its previous development; but through time, similarities develop and ultimately all basins experience similar problems of siltation. The principal differences between the ways in which infilling is achieved are dependent upon the nature of the river-supplied sediments and the thermally or atmospherically induced circulation patterns in the waters.

The most obvious visible effect of the impoundment is to raise water levels, flooding influent tributary valleys, causing aggradation in the valley bottom. Deposition on the valley floors extends to all influent river mouths and delta growth ensues. The delta plan is dependent upon basin shape and wave reworking, giving leveed, or arcuate deltas in the extreme cases of quiet and agitated waters respectively. The vegetated coarse sediment of the delta tops becomes progressively finer into the basin and as the materials are deposited at their angle of rest, they may slide or slump into the basin from time-to-time.

Around the reservoir margins, wave activity creates erosional cliffs and depositional beach berm and slope assemblages, and the stability of the water level determines the maturity achieved by the beaches. Inlets become silted through longshore sediment migration and locally spits and bars form prominent features.

The main influent rivers and distributaries provide channels crossing the subaqueous delta slopes, routes along which underflows pass into the main basins. The turbidity flows carry large quantities of sediment which are also deposited in the topographically lowest sites along the former channels as far as the dam, or within former basins, raising the bed level towards a fairly uniform flat sediment surface which covers the majority of the basin floor. The nature of the deposits, the presence or absence of layering and the strength of the deposited materials depend strongly upon the processes of deposition and the circulation patterns giving rise to settling.

Overflows and interflows also carry suspensions across the reservoir at levels determined by the relative density of the inflow and the thermal density structure of the water. Settling of materials from suspension either concentrated at the thermocline or more dispersed during isothermal conditions produces a general mantle of "pelagic" sediment to all submerged features, whether natural or artificial. Thus, drowned tree trunks, farm houses, roads, fields, drainage ditches, etc. are equally covered. Where pelagic sedimentation is slight, features like walls may be recognizable for scores of years, but rapid buried produces a general flat or gently undulating surface throughout. Thirty years after flooding, the western influent delta end of Loch Tummel, Scotland, still preserves virtually unaltered the texture of the flooded agricultural land, walls, fences, and tracks, and grab sampling reveals soils and dead grasses covered by very thin deposits of silt. Elsewhere siltation is more rapid. In the White River Reservoir of Wisconsin, U.S.A., Batten and Hindall (1980) recorded that in a 69-year period most of the early geomorphological features were buried, with silt thicknesses of over 5 m along the former river channel and over 8 m in the deepest part of the sediment pool beside the dam. The originally irregular morphology of the reservoir had undergone substantial simplification (Figure 2.8).

As morphological simplicity increases so the influence of physical irregularities within the basin decreases and the water circulation patterns become modified, leading to changes in sediment distribution patterns. Thus, an originally complex geomophological system, with variety of sediments and physical controls progresses towards a uniformity of sedimentation and simplicity of basin form which may itself break down as tributary deltas build out across the basin. Reservoir operation techniques leading to drawdown and systematic scouring of the sediment pool above the dam also contribute to the redistribution of sediment.

During period of substantial drawdown exposure of the deposited sediments leads to their draining and compaction. Two phenomena are involved, namely, compaction, which is largely permanent, and shrinkage, due to loss of water from the structure of clay minerals such as smectite and illite, and this is usually recoverable upon submergence. Measurement of polygonal shrinkage cracks on the floor of the Haweswater Reservoir (England) suggested that 12-16% lateral shrinkage of the surficial material was normal (Donovan and Archer, 1975). Similar estimates were obtained from the drawndown Glenfarg Reservoir (Scotland), where estimated shortening of the sediment column by 5-10% occurred (McManus and Duck, 1983 in press). Thus, the compaction of deposited sediments upon exposure to the atmosphere makes a small contribution to the recovery of storage capacity lost through sedimentation.

2.6 SHORELINE EROSION

When water surface level is at its maximum the wave attack leads to the removal of debris from the cliff-foot zone and its redistribution offshore. As Carstens and Solvik (1980) pointed out, if each wave achieves only very small changes, the end result of a year of such wave activity would show cumulative effects of some significance. Although only a fraction of the waves normally induce erosion, over a period of years changes are produced. The cliff-foot erosion causes shoreline retreat through collapse of unstable undercut slopes, especially in wet weather and leads to the growth of reservoir margin beaches.

Failure of waterside shorelines through generated landslides is marched further into the reservoir during drawdown below the normally occurring water levels. Rapid removal of the stabilizing water pressure reduces the safety factor of the slopes so that serious sliding may result. The problem is partly related to escape of the pore waters from the deposited sediments, and the effects may be minimised by ensuring that water level reduction is achieved slowly.

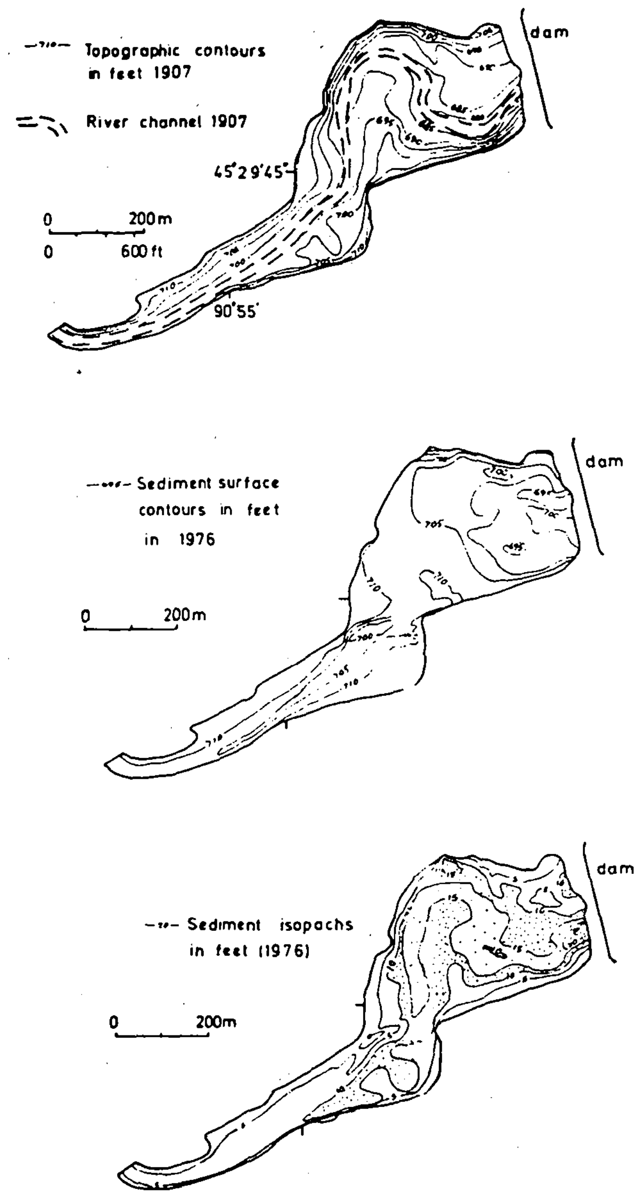


Figure 2.8. Sediment deposition in the White River Reservoir (After Batten and Hindall, 1980)

Removal of the restraining waters also permits escape of groundwater at and above the migrating shoreline. Spring line retreat throuh induced washout by the headwater rills of the new streams not only erodes the floor and slopes but locally steepens slope angles and stimulates landsliding.

Once the water level is reduced the influent streams flow to a lowered base level, so that degradation of their beds is achieved.

In high latitudes winter lowering of temperatures may lead to the development of an ice cover in the reservoir. This prevents wave attack on the shoreline, but during the thaw floating ice blocks may incorporate shoreline sediment and carry it away to the deeper parts of the reservoir before melting. Deep channels may be cut by icefloes and icebergs as wind drives them across the water surface. The ice may thus erode shoreline areas, scar the reservoir sides and floor and induce subaquatic sliding on steep slopes, in addition to transporting rock and sediment into the main body of the reservoir.

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Chapter 3
Field Measurements

3.1 FIELD MEASUREMENT OF SEDIMENT INPUT AND DISCHARGE FROM LAKES AND RESERVOIRS

Measurement of the input to, transport through, and discharge of sediment from lakes is the subject of the section. Sediment is carried into a lake by streams and rivers as well as by overland flow entering a lake. Sediment entering a lake may consist of a wide range of sizes, from gravel or boulders to silt and clay particles. Because of the low current velocities available to transport the sediment through a lake, the coarser particles are quickly deposited as the water enters a lake to form deltas. Seldom will any sediment larger than silt size be discharged from a lake.

The surest way of obtaining an accurate determination of the amount of sediment being carried to a lake by streams is to measure the flow rate and sediment concentration of the inflowing waters just upstream of the lake. Although some general considerations in sediment sampling are outlined below, and some of the newer sediment sampling equipment is mentioned, a thorough discussion of sediment samplers and sample collection techniques is beyond the scope of this report. The reader is referred to the following reports: World Meteorological Organization (Rapporteur, 1981), United States reports prepared by the Federal InterAgency Sedimentation project (Inter-Agency Committee on Water Resouces), and the U.S. National Handbook of Recommended Methods for Water-Data Acquisition (Guy, 1978).

For purposes of measurement, the sediment carried by a stream is usually subdivided into two parts: (a) fine: with particle diameters less than 0.062 mm and (b) coarse: with particle diameters greater than 0.062 mm that can be further divided into suspended load and bedload. Fine sediment is easily suspended by stream turbulence and travels at about the same velocity as the water. The concentration of fine sediment in the water depends on the amount supplied by the drainage basin and is usually associated with rainfall. The concentration depends only indirectly on the flow rate or carrying capacity of the stream. Since the concentration of fine sediment can be highly variable with time (variations by a factor of 100 to 1000 are common) much of the error in sampling results from the lack of enough observations to define the large temporal variations.

Measurements of fine sediment concentrations usually involve collecting a sample of the water-sediment mixture, separating the sediment, and weighing. Most samplers are operated manually. A problem with suspended sediment samplers is that they usually collect a small sample. Although some of the newer electronic instruments for measuring size distributions of fine sediments require only a small sample, most traditional methods for measuring size distributions of fine sediment and all methods applicable for coarse sediments require a relatively large amount of sediment. A sampler developed by the Jaroslav Cerni Institute consists of a vacuum tank and suction pipe with a nozzle mounted on the weight of a current meter (Miloradov, 1968). Sampling is done simultaneously with velocity measurements and a sample volume of more than 10 L can be obtained. In some cases, pumping samplers are used to collect samples automatically. These samplers are intended for use in permanent installations where the intake can be fixed at a given point. Samplers pump the water-sediment mixture through a tube and discharge it into a rack of collection containers. The PS-69 pumping sampler, for example, is especially applicable to sampling fine sediment. It operates on storage batteries and pumps a water-sediment mixture from a fixed intake point in the stream according to a predetermined scheme (Guy, 1978, p. 3-22). The sampling procedure can be initiated with a manual switch, a constant interval timer, or a switch that activates a timer whenever a preselected stage is exceeded.

Monitoring devices which sense the fine-sediment concentration continuously, or at least every few minutes, have been developed. Instruments have been developed which sense light attenuation or scattering, nuclear radiation, or sound (Guy, 1966). Unfortunately, none of these devices have proven acceptable for all conditions (Guy, 1978, p. 3-28). For special applications, however, some instruments may work very well.

While fine sediment is easily suspended by stream turbulence, coarse sediment has a strong tendency to settle to the channel bottom. Depending on the particle size and stream turbulence, coarse sediment can be mixed rather uniformly in the vertical or be extremely concentrated near the bed. The part of the coarse sediment, the smaller particles, which tends to be carried in suspension along with the fine sediment is called the suspended load. Other particles, especially the larger ones move by rolling, sliding, or hopping in almost continuous contact with the bed. This material composes the bedload. Although the U.S. Geological Survey has observed bedloads as small as one percent of the total load on the Tanana River and as large as 33 percent of the total load on the East Fork River (Emmett, 1983, personal communication). Linsley and Franzini (1979, p. 159) states that bedload generally accounts for between 5 and 25 percent of the total coarse material transport.

Coarse sediment is usually available in abundant supply on the riverbed. The rate of its transport, therefore, is governed by the ability of the flow to move the material rather than the supply as was the case for fine sediment. The rate of transport of coarse material is related to the water discharge but tends to be highly variable both with time and position in the river.

The suspended load can be sampled using procedures similar to those for fine sediment except for two special considerations. First, because of the inertia of the coarse particles a representative sample can only be obtained if the intake velocity to the sampler is equal to the flow velocity at the point of measurement. The second consideration which must be considered is that the suspended coarse material tends to have a large spatial variability, especially in the vertical direction. A single sample, therefore, will seldom be representative of the cross-sectional average concentration. Bruk et al. (1981) indicate that the simultaneous measurement of concentration and velocity is important to obtain a good correlation of the measured and theoretical concentration distribution.

Measurement of the bedload discharge is extremely difficult. Usually, the measurements have been made with samplers. Unfortunately, the amount of material transported as bedload is extremely sensitive to the local water velocity and no physical sampler can be placed in the flow without disturbing the water velocity near the sampler. Nevertheless, bag type samplers such as the Helly-Smith (Emmett, 1980, Rapporteur, 1981, p. 37) are probably the most commonly used. At present, no single sampler or type of sampler can be recommended. Other techniques are sometimes employed. For instance, bedload discharges have been determined by use of sediment traps (Emmett, 1980), from measurements of the migration of bed forms (Simons et al., 1965), the movement of tracer particles (Sayre and Hubbell, 1963, Tool, 1976), the erosion or deposition in a given area, and the difference in the concentrations of some nonconservative property associated with the bedload particles between different locations along the path of particle movement (Hubbell and Glenn, 1973). Whereas reasonably accurate results have been obtained with these latter methods, they all depend on information collected over relatively long periods of time. It is also common to estimate the bedload transport using one or more of various formulae based on sediment properties and flow hydraulics.

The total sediment transported (total load) by a river consists of the sum of the suspended fine sediment, the suspended coarse sediment, and the bedload. Because of the different modes of movement of each component, no one sampler is available for sampling the total load at a natural section of a river. The total load is usually determined by combining separate measurements of the suspended load (both coarse and fine) and the bedload. In some unusual cases such as outfalls, weirs, or highly turbulent sections, the flow may suspend all sediment and conventional equipment can be used to measure the total load. These procedures have been described by Vanoni (1977).

Because of variation in velocity, depth, bed form, etc., the sediment transport rate is usually quite variable in the lateral direction across a river. Vertically averaged concentrations of suspended sediment have been found to differ from one side to the other of the Mississippi River by as much as 2,400 mg/L and at many times the difference was greater than the mean concentration for the cross section (Vanoni, 1977, p. 323). For this reason, sediment discharge measurements in rivers must be conducted much like water discharge measurements. That is, the vertically averaged sediment concentration is measured at several verticals across the river and these concentrations are multiplied by the discharge assigned to the vertical to determine the sediment flux through each subsection. The individual sediment fluxes are then summed to obtain the total sediment discharge of the river.

Ordinarily, records of sediment discharge (fine, coarse, suspended, or bedload) are determined on the basis of sample information obtained at non-uniform intervals of time. The sediment samples are usually obtained by observers at a frequency which is low in relation to the characteristic frequency at which sediment concentration in the river changes. The information from the sediment samples can be used to estimate the total sediment discharge by developing a relation between the sediment discharge and water discharge. This relation, called a sediment transport curve, can be used to estimate the total sediment discharge of the river from the corresponding continuous flow record of the stream. The relation between sediment discharge Q_s, and streamflow, Q, is often expressed mathematically by an equation of the form

$$Q_s = a\ Q^b$$

where b commonly varies between 2 and 3 while a, the intercept at Q = 1, is usually quite small (Linsley and Franzini, 1979, p. 159).

For many streams, where a fairly stable relation exists between velocity and depth, a relatively stable relation also exists between the coarse sediment transport and the flow (Guy, 1978, p. 3-28). In these cases, the sediment transport curve should give a reasonably accurate estimate of the total coarse sediment transport. The rate of soil erosion, on the other hand, varies with rainfall intensity, soil conditions, vegetative cover, and many other factors. So although the concentration of fine sediments may also appear to be correlated with flow, the relation is usually not as precise as the one for coarse sediment. In some cases, the sediment transport curve shows a definite seasonal shift so that separate curves must be developed for each season (Strand, 1977, p. 769).

A common problem with sediment transport curves as well as monitoring sediment transport in general is that extremely large sediment loads are usually associated with floods. At these infrequent times, when sampling is most difficult, a significant portion of the total sediment transport may occur. As a result, the most important part of the sediment transport curve is often poorly defined because of inadequate sampling during large floods.

Several years of record are often needed to develop reliable information concerning average annual sediment transport. For a given degree of reliability, the length of record needed will depend on the number of sediment producing events each year, the frequency of sampling, and the range in sediment conditions encountered among the different events. Data of Neff (1967) illustrates the problem. Neff found that 60 percent of the longterm sediment yield in arid regions was associated with runoff events having a return interval of 10 years or more. In humid regions only 10 percent of the sediment yield resulted from these larger floods.

Sediment is transported into lakes, reservoirs, and ponds either as suspended sediment or as bedload. In the upstream part of a reservoir, data may be collected on sediment movement much as the data are collected in a river because of the appreciable flow velocity. As the sediment-laden water travels further into the reservoir, however, the velocity and turbulence are greatly reduced. The bedload and the larger suspended particles are quickly deposited to form a delta at the head of the reservoir. Smaller particles remain in suspension longer and are deposited further downstream. As the water-sediment mixture progresses into the reservoir, the flow velocity approaches zero, and movement, if any, results from complex circulation patterns. The smallest particles may remain in suspension for a long time and some may pass through the reservoir and be discharged through sluiceways, turbines, or the spillway. Because of the complex circulation patterns and the complexity of the suspended sediment movement, sampling techniques need to be designed for each individual case based on the objectives of the sampling program.

3.2 RATES OF ACCUMULATION

3.2.1 Surveying Methods

The ultimate fate of any reservoir is to become filled with sediment. The useful life of a reservoir is the time from construction until it is filled with sediment to a point where it no longer serves the intended purpose. Because the rate of sediment accumulation in reservoirs cannot be accurately predicted, it is almost always desirable to determine the volume and weight of sediment accumulated during specific intervals of time during the entire life of the reservoir. This information may be needed to:

1. Estimate the sediment yield for given watersheds or land resource areas,

2. Evaluate sediment damage,

3. Provide basic data to plan and design other reservoirs,

4. Evaluate the effects of watershed protection measures,

5. Determine the distribution of sediment in a particular reservoir, and

6. Predict the life expectancy or period of useful operation of a reservoir.

Upon completion of the construction of a dam to impound water, regardless of the size, a plan to monitor the reservoir should be established. This plan may vary widely depending on the size, operation, purpose or quantity of sediment inflow expected. Some discussions helpful in developing a realistic plan and schedule for reservoir surveys are contained in Guy, 1978, p. 3.57-3.58; Vanoni, 1977, p. 349-350; and Pemberton and Blanton, 1980, p. 1-3. 1977, p. 349-350; and Pemberton and Blanton, 1980, p. 1-3.

The general procedure for making reservoir surveys has changed little since it was described by Eakin (1939). However, there have been significant advancements in the equipment available to execute the basic procedures. Basically the general procedure is to construct a bathometric map of the lake bottom which can be compared to a previously constructed map to determine differences in the volume of sediment deposited. There are two general methods of conducting the reservoir survey. These are the range-line survey and the contour survey. Selecting the method depends on the availability and character of previous mapping or survey records, the purpose and scope of study objectives, the size of the reservoir, and the degree of accuracy required.

The range-line method is the most widely used for medium to large reservoirs requiring an underwater survey utilizing hydrographic surveying methods. To apply the method a number of cross sections of the reservoir are surveyed before it is first filled and then periodically resurveyed. These cross sections are called ranges. The range-line method usually requires less field work and is less expensive than the contour method. On the other hand, it also is usually less accurate. Specific details concerning the method can be found in many references including Vanoni, 1977, p. 369-382; Guy, 1978, 3.59-3.82; and Eakin, 1939.

The contour method uses essentially topographic mapping procedures (Wolf, 1974). The method is especially suitable for aerial surveys when flights can be scheduled for different known pool elevations. To apply the method it is important to have a good contour map of the reservoir before filling. The contour method is usually used for small reservoirs, reservoirs which are occasionally empty or at low stage, or when the highest degree of accuracy is required.

Selection of the contour interval is controlled by the same factors which are used in selecting a map contour interval, but it is suggested that the interval not exceed 1.5 m and 0.5 m for large and small reservoirs respectively (Pemberton and Blanton, 1980, p. 2).

The basic procedure for either method involves the determination of bed elevation at many known locations on the reservoir. These measurements are almost always made by measuring the water depth beneath a boat and the exact location of the boat on the lake's surface. So two basic types of measurements are required, position measurements and depth or bed elevation measurements. The basic procedures and equipment selection considerations have been presented by Vanoni, 1977; Hart and Downing, 1977; and Guy, 1978. Selection of the most appropriate equipment depends on many considerations.

The simplest way of measuring the water depth is to use a sounding weight or a pole to obtain it directly. If a sounding weight is used, the weight and shape of the sinker should be accurately recorded so that later surveys can use the same type weight. Otherwise results may not be comparable in areas where the bottom deposits are soft.

Sonic sounding equipment for measurement of depth is preferred on most reservoirs. The scientific depth sounding equipment currently available can be used to provide a continuous record or chart of the bottom profile. The basic components are a recorder, the transmitting and receiving transducer and a power supply. By careful calibration, a high degree of bottom profile accuracy can be maintained. Sonic sounding equipment can be relatively inaccurate in situations where the bottom slope is extremely large (Vanoni,1977, p. 353). 1977, p.353). An ordinary sonic sounder operates with a signal frequency of 60 KHz which is quite acceptable for the detection of the water-bottom interface when the bottom is composed of sand or gravel. For a very soft muddy bottom however, it might indicate the interface is 10-15 cm deeper than the true value (Rakoczi, 1984, personal communication). Sonic devices with about 120 KHz frequency can solve this problem and give some information about the underlying strata, however the interpretation of the results is often difficult due to the poor degree of resolution. Through skilled interpretation those records may provide useful information related to particle size, degree of compaction, rate of deposition and other desirable characteristics of the bed.

Mr. Laszlo Rakoczi (1984, personal communication) feels impulse radars are superior to sonic devices because the former are especially built for detecting subsurface soil layers with much better resolution than sonic devices. The proper name for these devices is Subsurface Interface Radar and they operate with electromagnetic pulses instead of sonic waves. They operate in the range of 300 - 1000 KHz (Rakoczi, 1984, personal communications).

The other basic measurement required for a reservoir survey is the location of the boat at the time the depth is measured. Many manual techniques have been used to determine the boat position. The simplest which is useful on small reservoirs is to use a tag line. For larger reservoirs the boat has often been located by triangulation methods using transits on shore.

Some of the most significant advancements in reservoir surveying during recent years has been the development of sophisticated navigation and positioning equipment. Marine positioning systems have used a wide variety of techniques in their operation, and over the years, many different user designations have evolved for the various types of equipment. Each designation may describe either a basic measuring element or an operating characteristic. In order to give the reader a quick summary of a number of the terms used for categorizing positioning systems, some widely used descriptors are tabulated. This tabulation developed by Hart and Downing (1977), attempts to show some of the relations between the alternate ways of describing systems. Those combinations in practical use are noted by an X on figure 3.1.

A major shift in technology use has occurred in the past few years as many surveying operations have shifted from manual to electronic techniques. Availability of commercial equipment within the categories listed in figure 3.1 has been presented by Hart and Downing (1977).

Active responder equipment includes those types of distance measuring equipment that have one unit to transmit an interrogation pulse and a second unit to receive the interrogation pulse and reply with an answering pulse, which can be decoded to derive range. Active responder equipment is widely used in the United States because of the large variety available (Hart and Downing, 1977, p. 11). Figure 3.2 contains a schematic of the operation of a typical modern hydrographic surveying unit. Passive responder positioning systems also use the distance measuring equipment to transmit signals but differ in that the response signal is reflected energy from the shore instead of a response transmission as with the active responder systems. Although many factors must be considered in determining surveying accuracy, (Hart and Downing, 1977), even simple systems often have a range of 20 km and an accuracy of ± 2m. Doppler navigators transmit signals and detect reflected energy similar to passive reflector systems, but the detection circuitry indicates the velocity of the moving vehicle and not distance. The displacement or distance is obtained by integrating the velocity over time. Doppler navigators will work without knowledge of the location of the reflective surface. Thus they offer great freedom of operation since there is no need to stay within range of shore responders or reflectors. The accuracy of a doppler navigator is inversely proportional to time from the starting point so the actual location must be periodically updated. Because the accuracy of inertial navigators is inversely proportional to time squared, they have seen little application in reservoir surveying systems. Satellites and astronomical readings do not provide sufficient accuracy for reservoir surveys at this time. Beacon positioning systems radiate optical, radio frequency, or acoustic energy from known locations, Measurement Medium or Coupling

Position Reference Technique	Nonline-of-sight			Line-of-sight					
	Very Low Frequency (VLF) 10.0 - 13.0 kh_z	Low Frequency (LF) 100.0 KH_z	High Frequency (HF) 1.6 - 3.3 MH_z	Ultra High Frequency (UHF) 0.3 - 3.0 GH_z	Microwave 3.0 - 10.0 GH_z	Optical	Acoustic	Mechanical	Magnetic
Active Responder			X	X	X		X		
Passive Reflector					X	X			
Doppler					X		X		
Inertial								X	
Satellite					X				
Manual						X		X	
Beacons	X	X	X	X	X	X	X		
Astronomical						X			X

Figure 3.1. Categories of positioning system techniques

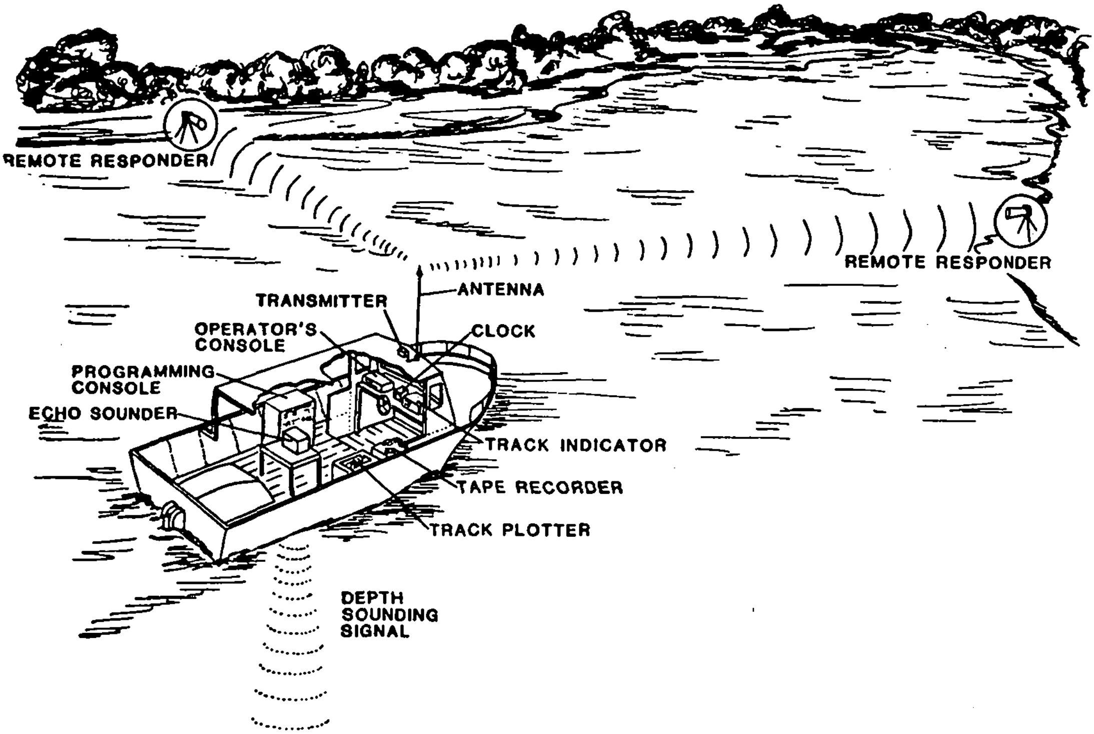

Figure 3.2. Schematic of the operation of a modern hydrographic survey

and the reference signal is available to all users with means to receive the signals. Radio beacons can be used to provide distance as well as azimuth references.

Except for the doppler and inertial systems all positioning equipment operate by sensing either the distance to two or more known reference points or sensing the distance and angle to a single reference point. It is generally easier to measure distances electronically than to measure angles. There can be either line-ofsight or non-line-of-sight systems, depending on the frequency of the signals used for measurement. Line-of-site systems operate much as the old manual methods with improvements in speed and accuracy.

The electronic systems are less labor intensive than the older mechanical positioning methods and are very adaptable to automated processing of the measured data. The equipment is usually very accurate but technically complicated and relatively expensive. Operational methods and procedures, which vary with the specific systems employed, are usually well documented in the manufacturer's operational and maintenance manuals. These systems have been assessed and discussed at several conferences on the subject (U.S. Corps of Engineers, 1972, 1973, 1974, 1982; and Hart and Downing, 1976, 1977, 1978, 1980).

Once the data, consisting of water depths and locations, have been obtained they can be used to compute reservoir storage volumes, sediment deposition rates or area capacity tables. The volume of the sediment deposits accumulated in a reservoir is generally obtained from the difference between the original capacity and the capacity computed from a resurvey. Many methods have been used to compute the reservoir capacity. Heinemann and Dvorak (1965) provide an excellent review of many of these methods, most of which can be programmed for machine computation.

The basic procedure to compute the capacity of a reservoir is an adaptation of the average end-area method commonly used in computing earthwork quantities. When the contour method of survey is used the computation procedure entails measuring the surface areas enclosed by each contour and computing the volume between successive contours. When the range-line method of surveying is used, the computation procedure involves determining the cross-sectional area at each range and computing the volume between ranges.

Perhaps the greatest advancement in recent years has been the development of on-board computers and data processing equipment. This equipment can be linked directly to the positioning and depth-sounding equipment to record the data, and in many cases plot the results, and provide pilot guidance. The complete system selection will be governed by (a) degree of field automation desired, (b) cost, and (c) boat size. Most systems for reservoir use should be small enough to be accomodated on a boat which can be towed on a trailer. These systems usually can provide pilot guidance and data recording capability but usually cannot provide finished plots. The system used by the U.S. Bureau of Reclamation has been very briefly described by Pemberton and Blanton (1980).

3.2.2 Dating of Sediment

The previous section was devoted to a discussion of reservoir surveys made to determine the rate of accumulation of sediments in a reservoir. The amount of sediment accumulation is essentially computed from the differences in bed elevations occurring between suveys. Unless the amount of accumulation is large, the accuracy of the procedure suffers because the desired result is computed as the difference of two large inexact numbers. In locations where the bed has a steep gradient, the error can be especially large because a small difference in position can cause a large difference in measured bed elevation. The U.S. Soil Conservation Service (Soil Conservation Service, 1973) recommends that the depth of accumulation should be measured directly unless it exceeds 0.3 m. In situations where a good initial survey is not available, the direct measurement of accumulated sediment is the only approach possible.

The usual methods to directly measure the amount of accumulation include polling, spudding, and coring. If the water is shallow and other conditions are favorable, the sounding pole is the easiest method to use. When the accumulated sediments are soft and the underlying natural material is firm, a sounding pole can be lowered to the sediment surface, the depth read and then the pole pushed down to the firm underlying material and the depth read again to determine the depth of accumulated sediment. When the uppermost layer of the bed is very soft the pole often penetrates unnoticed for several centimeters. A cheap and simple device to correct this problem has been described by Rakoczi (1983). A spud rod consists of a steel rod with grooves in it designed to trap sediment material. The rod is connected to a rope and dropped vertically into the sediment. Upon retrieving the rod, the sediment trapped in the grooves is inspected to determine the thickness of the new sediment. To be effective, the spud rod must completely penetrate the accumulated sediment and the accumulated sediment must be distinguishable from the underlying natural material. Core samples are more difficult to obtain but provide a better sample of material from which to determine the interface between the natural and accumulated sediment.

A lake receives runoff and sediment from its drainage basin and most of the sediment is trapped on its bed. Because all hydrologic processes are interrelated to some degree the trapped sediment contains a record of the physical, chemical and climatic conditions existing over the basin. Any significant change in these conditions should leave a record in the accumulated sediment. If these changes can be identified and dated in the accumulated sediment, the rate of sediment accumulation can be estimated. Natural tracers of various types have been used to determine rates of sediment accumulation in reservoirs over time scales ranging from decades to thousands of years (Winter and Wright, 1977).

Carbon-14 dating of organic sediments has been used to estimate sedimentation rates during past millenia. The method, which is discussed in appendix 3.1, is not appropriate for relatively recent times of practical interest in problems of sedimentation in reservoirs. Robbins and Edgington (1975) used the activity of ^{210}Pb to estimate the sedimentation rate in Lake Michigan, U.S.A. on on the basis of cores from eight locations which indicate that rates over the past hundred years or so have been constant. The method was found to be in good agreement with an estimate based upon the distribution of pollen.

A relatively easily detected tracer was introduced worldwide by the test programs for nuclear weapons beginning in 1954. Cesium-137, which does not occur naturally, was produced in these tests. Cesium-137 has been deposited on the earth as fallout and since the fallout has varied with time, there should be a similar variation in the ^{137}Cs content of a sediment profile. A profile of ^{137}Cs concentration against depth should show a maximum activity at a depth which corresponds to 1963 and no ^{137}Cs for sediments deposited before 1954. A more detailed description of the method is contained in appendix 3.1.

Robbins and Edgington (1975) also studied the distribution of ^{137}Cs in the cores of Lake Michigan, U.S.A. and suggest that the use of both the Cesium-137 and the Lead-210 methods is to be recommended, since the combined approach can provide a better insight to the physical and chemical nature of the sedimentation process.

Most natural tracers probably occur on a more localized area. A change in cultivation or environmental conditions in a basin may provide an identifiable and unique tracer. For example, a distinct increase in ragweed pollen occurs in the sediment deposited in lakes of the central and eastern United States at the time of first settlement. This natural tracer provides good indication of sediment accumulation during the past 100 years or so. If the introduction of a specific crop to an area can be dated then the distribution of the pollen in sediments may provide a record of sedimentation. For reservoirs receiving sewage it is often possible to detect a rise in the phosphate concentration for sediment deposited after the introduction of phosphate-based detergents. Bradbury and Megard (1972) found a sharp increase in hematite and limonite concentration corresponding with sediments deposited in Lake Shagawa, Minnesota, U.S.A. after 1900 when mining began in the basin.

Although a great variety of materials are preserved in lake sediments, those that have been studied in detail include mineral materials such as sand, silt, clay and marl; biological matter such as pollen grains, diatoms, mollusks, Cladocera, charcoal, seeds, and other plant or animal microfossils; and chemicals such as amino acids, pigments, and a wide variety of inorganic elements as well as such nutrient elements as phosphorus and nitrogen (Winter and Wright, 1977). Whether or not the sediments of a particular lake or reservoir contain a definable shift in the concentration of any of these materials will, of course, depend on the history of the specific basin. But if a horizon can be identified and associated with a historical event, a fairly accurate determination of sediment accumulation should be possible.

3.2.3 Remote sensing

Low level remote sensing has several main applications in the assessment of reservoir sedimentation. Contour maps prepared from aerial photographs can be used to determine sediment volumes provided the water level can be lowered greatly; aerial photography can be used to trace turbidity plumes which may help define the distribution of sedimentation, and airborne laser hydrography is being tested.

Recent technological developments in precision mapping photography have lowered its cost relative to standard hydrographic surveys. The U.S. Bureau of Reclamation considers photogrammetric surveying to be the most cost effective method of conducting a survey of reservoir capacity providing the water level can be drawn down to a very low level (U.S. Bureau of Reclamation, 1975, p. 11).

The largest cost (53 percent of total) in conducting a resurvey of Gibson Reservoir Montana, U.S.A., was for providing the horizontal and vertical control (U.S. Bureau of Reclamation, 1975, p.8). This control included ground panel targets which consisted of white colored material in the form of a Red Cross symbol having an overall size of 4 m by 4 m. Aerial photography represented only 9 percent of the total cost while the preparation of the contour maps represented 38 percent of the total cost.

In situations where providing accurate vertical control is difficult, non-stereo aerial photography has been used with the reservoir pool at different levels. The shoreline then defines a contour and a contour map can be constructed by superimposing the results of several flights.

Knowledge of the location and extent of turbidity plumes is helpful in predicting the sediment distribution patterns in reservoirs. Aerial photography has been used to study turbidity plumes in rivers (Scherz et al., 1979) as well as estuaries (Gatto, 1976, 1980). Unfortunately, remote sensing can only sense surface plumes so the method has limited applicability in reservoirs where the turbidity plumes quite likely will occur as interflows or underflows. Although the results are only qualitative, it may be possible to determine which inflows to a reservoir are contributing the greatest sediment load by use of remote sensing during a flood. Gatto (1980, p. 299) found color photography to be more useful in detecting sediment plumes than color infrared because the exposure setting is less critical and because it has greater water penetration.

Some investigators have found radio controlled aircraft to be useful and cost effective in obtaining aerial photography. A craft with a 2 1/2 m wing span can carry a 35 mm camera to a height of 300-400 m. These planes can take off and land on the lake surface and typically have a range of about 3 km. Helium filled balloons have also been used as a camera platform and the recent development of ultra light aircraft presents other interesting possibilities.

A new technique which is still only being developed by the U.S. National Ocean Survey is to use laser hydrography (Enabnit et al., 1979). The airborne laser hydrographic technique uses an aircraftmounted, pulsed laser system to collect a swath of discrete soundings along each flight line. It measures water depth exactly like a sonar using light instead of sound. The National Ocean Survey operating system will take 600 soundings per second over a 200 m wide swath with an average distribution of 1 sounding per 25 m^2. The system will operate from a light plane flying at 300 m with a speed of 75 m/s.

The major benefit of the system is its ability to generate many soundings in a very short time. For example, Enabnit et al. (1979) estimate that a single system could survey more area annually than the entire U.S. National Ocean Survey launch fleet and also provide a 300-fold increase in the number of soundings per unit area. Its accuracy seems to be about 0.15 m. Because it is an optical technique the depth of penetration will be limited by water clarity. Along the Mid-Atlantic coast of the United States, Enabnit, et al. (1979) expect the system to operate in water depths of about 20 to 30 m.

Although the system is being developed for use in the coastal environment, its potential application to reservoir surveying is obvious.

3.3 SEDIMENT DENSITY

The bulk density of a sediment deposit is the dry weight of sediment per unit volume. The bulk density, therefore, provides a simple and direct conversion from the dry weight of sediment added to the reservoir to the volume of water displaced. The bulk density depends primarily on the particle size of the sediment but this relation may be altered by the sorting of grain sizes. In addition, organic material can sometimes make up a large part (up to 50 percent or greater) of the lake sediments. Sediment also tends to consolidate with time due to overburden pressures as well as to the alternate wetting and drying if water levels fluctuate widely. It is desirable, therefore, to measure the bulk density of sediments in reservoirs.

Traditionally these measurements have been made by use of core samples. A variety of types of samplers have been used to obtain undisturbed samples of reservoir deposits for determining the bulk density and/or grain size distribution. The type of sampler to be recommended varies depending on such factors as sediment grain size, depth of water, degree of consolidation, and sediment depth. Excellent summaries of available equipment have been presented (Guy, 1978, p. 3-40).

Probably the greatest advancement in this equipment in recent years has been in the use of plastic liners which aid in maintaining the sample in a relatively undisturbed state even for relatively low density samples (Vanoni, 1977, p. 358). An instrument which was especially designed to obtain reliable information on density of reservoir deposits has been described by Milisic (1981). It consists of a cylinder which is pushed into the deposit and the sample is frozen. The sample can then be brought to the surface in a solid state.

Sometimes, in design for example, it is necessary to estimate density of reservoir sediments without the benefit of in place measurements. Empirical relations have been developed between bulk density, sediment size and time after deposition. Revised coefficients for one of these relations have been presented by a Federal Interagency Committee of the United States (Guy, 1978, p. 3-44).

The traditional method of measuring bulk density by a core sampler is laborious and time consuming. The use of nuclear density probes has therefore received wide acceptance. The response of a radiation detector to scattered or transmitted gamma radiation from a radioactive source is a function of the density of the sediment. Measurements may be made in situ or on core samples.

In the case of a gauge operating on the scattering of radiation, the probe unit contains a source of radiation and a detector. The detector counts the impulses from the radiation scattered by the sediment in a spherical region of about 40-50 cm diameter around the probe (Rapporteur on Sediment Transport, 1981, p. 48). A typical design used at the Institute of Oceanographic Sciences, U.K. (Parker et al., 1975) consists of a NaI(Tl) crystal scintillation detector located at a distance of 20 cm from a 1 mCi source of ^{137}Cs which is contained in the end of the probe. The response of the detector to direct radiation from the source is minimized by the presence of intervening shielding. The unit may be operated on 12 to 24 volts DC or on 220 volts AC. Prior to use in the field the probe is calibrated, so that the readings from the rate meter connected to the detector may be interpreted in terms of bulk density. The gauge response is also a function of the composition of the sediment, so that calibration should be carried out with material of similar composition to that of the sediment to be measured. The accuracy of measurement by gamma-scattering probes is $0.02 \times 10^3 kg\ m^{-3}$.

Measurements on core samples are preferably made with a gauge operating on the transmission principle. With a well collimated source of radiation it is possible to obtain a profile of density measurements on a core. However, whereas the high energy of a ^{137}Cs source limits the effects of variations in chemical composition, it does render the measurements insensitive to small variations in density. If the latter is of importance then it may be advisable to use a radio active source of lower gamma energy, but this will necessitate greater attention to corrections for effects due to variations in chemical composition of the sediment.

If transmission is detected in situ, the source and detector are mounted in separate tubes a known distance apart. The distance between the tubes must be selected based on preliminary experiments depending on the energy of the isotope source, the compactness of the sediment, the sensitivity of the detector, etc. The advantage of the equipment which detects transmission is that the density in narrow layers (2-3 cm thick) can be measured and the deposit stratification can be detected. The disadvantage is that two tubes are required and these tubes must be inserted into the sediment vertically and precisely parallel. McHenry (1971) describes such a probe which is available for use in shallow reservoir deposits.

3.4 PARTICLE SIZE

Particle size information is useful in predicting erosion, transport, deposition and compaction of sediments. The size distribution of incoming sediment is very important in determining the pattern of deposition in a reservoir and is one of the most important aspects in the evaluation of the amount of space a given weight or quantity of transported sediment will occupy in a reservoir. As more emphasis is placed on water quality, it is necessary to measure the fine silt and clay portions of the size distributions in much greater detail because of their importance in the trapping and transport of chemicals such as nutrients and pesticides.

An extremely wide range of sizes of sediment particles can be supplied to a reservoir. The large and intermediate sizes (gravel and sand) will be deposited almost immediately upon entering the system. These particles can be analyzed for size using standard techniques which will not be discussed here.

The small sizes (clays, silts, and perhaps fine sand) are of primary interest. The sizes of these sediments are generally measured indirectly by observing their settling characteristics in water. Traditional sedimentation methods most commonly employ the pipet, the bottom-withdrawal (BW) tube, and the hydrometer. The pipet method is considered the most reliable method for routine use to determine the particle size gradation of fine sediments (less than 0.062 mm). Traditional sedimentation methods for size analysis have been described in detail (Vanoni, 1977; Guy, 1969) and so they will not be discussed further here.

In the past few years, several electronic instruments have been developed for measuring size distributions of fine sediment. One of the first such instruments, the Coulter Counter (Coulter, 1956; Berg, 1957) detects the change in electrical conductivity as particles pass through a small orifice. The output pulse is proportional to the volume of the particle. This instrument can be used for sediment sizes between 0.0003 mm and 0.3 mm. A sample can be analyzed in 5-10 minutes and detailed data output in either graphical or tabular form is produced (Schiebe et al., 1981).

Other instruments detect the intensity and scattering angle of laser light light by particles in a sediment suspension (Wertheimer et al., 1978; Haverland and Cooper, 1981). These instruments are very simple to use, require about 6 minutes to analyze a sample and can operate in a size range of 0.0019 - 0.176 mm. The output is in tabular form providing summary data as well as both differential and cumulative results (Schiebe et al., 1981).

Some electronic instruments use the principle of gravitational settling of particles in a small cell (Oliver et al., 1970; Welch et al., 1979). The principle of operation is similar to the pipet method except the concentration in the cell is detected by a weak x-ray beam 0.050 mm thick (Schiebe et al., 1981). This instrument operates within the range of 0.0001 - 0.1 mm and requires about 15 minutes to produce a graphical output in cumulative form (Schiebe et al., 1981).

All the electronic systems are very expensive but convenient to operate. The results of different instruments cannot always be directly compared with each other or with the pipet method (Schideler, 1976; Schiebe et al., 1981). Allen (1981) presents more detail on the measurement of particle sizes in sediment.

3.5 WATER CURRENTS

As river water enters a reservoir, the velocity rapidly decreases and all coarse sediment is deposited, usually forming a delta. The very fine sediment can remain in suspension for a long period of time however, and is carried to all parts of the reservoir by internal currents. Water currents in a reservoir are caused by wind, density differences and throughflow of the river. In order to predict the distribution of the deposition of incoming sediment or to estimate the trap efficiency of a reservoir, it is frequently desirable to measure the velocity and water flux of reservoir currents. Current meters, drogues, and tracer techniques have been used for these measurements.

The use of a current meter would appear to be the obvious method of measuring water currents. However, unless very high sediment concentrations produce a strong density current, the low velocities involved (generally less than 0.1 m/s) and the difficulty of providing a stable meter support, have made current meter data unreliable. Although a considerable amount of effort has been expended, little progress appears to have been made in developing a reliable current meter for measuring low velocity lake currents.

Because of vertical variations in density due to temperature, salinity or sediment concentration, the vertical distribution of velocity in a reservoir can be very complex. Drogues work reasonably well for tracking surface currents, especially when combined with aerial photography. They are less effective at tracking underflows, however, because of the difficulty of predicting the depth at which the maximum current will occur. They have been used, nevertheless, and in some cases their position has been tracked by radar.

Fluorescent dyes have been used to track currents in reservoirs with some success. To apply this technique a mass of water is tagged by use of a fluorescent dye and the location and spreading of the dye mass is tracked with time. In the past few years, the portability and sensitivity of fluorometers, used to detect the dye concentration, have been increased significantly. With modern fluorometers, it is possible to detect dye concentrations as low as 1 part in 10^{11} parts water (Smart and Laidlow, 1977). Improvements have also been made in the properties of dye available for use. Modern dyes (such as rhodamine-WT) have less tendency to absorb on sediments than older dyes. Combining the portability and sensitivity of modern fluorometers with modern positioning equipment should make it possible to measure water currents with reasonable accuracy and ease. Keefer and McQuivey (1980) report the results of a study where fluorescent dye was used to determine the travel time and path through a reservoir as well as the rate and nature of lateral mixing.

A special case exists when very high sediment concentrations occur such as during flood inflow. In these cases, such a strong density current is formed that current meters give satisfactory measurements of velocity. It is usually desirable to sluice as much as possible of the sediment in this current out of the reservoir,so it is necessary to monitor the movement of the plume through the reservoir. Field measurements are usually obtained to monitor the movement of the density current and to estimate the portion of the sediment which is sluiced out. To fulfill these goals, current data must be obtained as the flood flow enters and leaves the reservoir as well as within the reservoir.

Measurements of inflow and outflow are similar to measurements in rivers in that profiles of velocity and suspended sediment concentration should be obtained at various times throughout the hydrograph. These data are integrated over the cross-sectional area to determine the inflow discharge of water and sediment. The temperature and salt content are likely to be uniform in the cross section but can vary with time. These data can be useful in delineating the plume in the reservoir and also are important in determining the density of the inflow water. The total sediment input to the reservoir, minus that sluiced out, is the quantity of sediment trapped by the reservoir.

In order to operate the outlet gates in an efficient manner and to study the movement processes of density currents, it is necessary to obtain data at various specified cross sections in the reservoirs. It is usually found that the density current follows the old river channel through the reservoir. At each cross section and at various times throughout the hydrograph, vertical profiles of velocity, sediment concentration, temperature, and dissolved solids content should be obtained at several verticals across the density current. In addition, the direction of the velocity vector, the elevation of the density current interface, and a size analyses of the suspended and bed sediment should be obtained. Since these high density plumes move along the bottom, the thickness of the density current and the elevation of the interface can often be detected by use of a sonic sounder with a high frequency signal.

3.6 RESIDENCE TIME IN RESERVOIRS

The average residence time for water in a well mixed reservoir is simply the volume of the reservoir divided by the inflow rate. Although this average residence time is a useful measure of the storage capacity and the relative importance of river currents to wind or density currents in the reservoir, it is not a good measure of how long a particular mass of water is likely to remain in the reservoir.

Most reservoirs, especially if they are deep or have a long average residence time, tend to develop a definite vertical density stratification structure. This vertical structure is a function of the vertical distribution of temperature, salinity and suspended sediment. The density of the inflowing water also varies with time due to the same causes. The inflowing water tends to seek a level at which its density and that of the reservoir are the same. Water entering a reservoir may spread out forming a thin layer on the top (overflow), plunge to the bottom (underflow), spread out in a layer at some intermediate depth (interflow) or mix fully over the depth depending on its density relative to the density distribution in the reservoir.

Outflowing water however tends to be withdrawn from a level corresponding to the level of the outlet structure. The thickness of the withdrawal layer is a function of the density variation, being narrow for highly stratified conditions and including the entire reservoir depth if no stratification is present. The thickness of the withdrawal layer can be approximated using methods developed by Bohan and Grace (1973). Therefore, depending on the level at which water is being withdrawn and the density structure, inflowing water may be short circuited through the reservoir rapidly, trapped for very long periods of time, or mix completely with the receiving water.

Generally speaking, analytic methods have not been developed which are capable of predicting the transit time of particular water masses through a reservoir. The most promising method for measuring the residence times of specific water masses is to use a tracer technique. The water mass can be identified by use of either natural tracers or artificial tracers such as dye as used by Keefer and McQuivey (1980).

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Chapter 4
Methods of Preserving Reservoir Capacity

4.1 METHODS OF MINIMIZING SEDIMENT DEPOSITION IN A RESERVOIR

4.1.1 Reduction of Sediment Inflow by Soil Conservation

Soil conservation methods in preventing the movement of soil particles or preventing the transport of sediment to the reservoir include watershed structures and land-treatment measures in the watershed.

Several types of structures may be built in a watershed, e.g. sedimentation basins to store sediment permanently for the design life of the reservoir or to store sediment for specific storm runoffs before periodic cleanout; drop inlets and chutes for reduction of gully erosion; stream-bank revetments to reduce stream-bank erosion; and sills or drop structures for stream-bed stabilization.

Watershed land-treatment measures to reduce sheet erosion include soil improvement, proper tillage methods, strip cropping, terracing, and crop rotations.

If the watershed is not very large, the effect of soil conservation can be felt in a short time. According to the experience in the United States, soil loss can be reduced by as much as 95 percent by employing no tillage rather than the conventional tillage (Holeman, 1980). But, if vast areas with poor natural conditions are involved, the soil conservation works can hardly be effective in a short period of time. The effectiveness of the soil conservation for large catchment areas cannot be estimated with accuracy.

Four case histories of soil conservation programs designed to reduce sediment transport are discussed. Two are in reservoirs, one in a river system, and the fourth relates experimental studies in a small gully watershed.

(1) The Tungabhadra Reservoir (Rajan, 1982)

The Tungabhadra Reservoir Project was constructed on the river Tungabhadra in India in 1953. The annual river flow varies from 8.4 km^3 to 17.1 km^3, the average being 11.47 km^3. Dry weather flow dwindles to a few m^3s^{-1}. The storage capacity of the reservoir is 3.75 X $10^9 m^3$. The catchment area at the dam site is 28,178 km^2, 14.5 percent of which is forest, 53 percent of heavy soils and 32.5 percent of erodible soils including black cotton soil. The project was planned assuming a rate of silting of 12.8 X $10^6 m^3$ per year (p.a.).

Sediment surveys were taken on the reservoir in 1963, 1972, and 1978. Table 4.1 contains the volume of sediment deposited for the various periods and this rate of silting expressed on a unit area basis.

It can be seen from Table 4.1 that the siltation rate was reduced after 1963. The reduction is partly attributable to trapping of sediments in reservoirs on the upstream tributaries and partly due to watershed treatment.

Table 4.1. Rate of siltation in Tungabhadra Reservoir

Period	Years	Volume deposited (10^6m^3)	Rate of silting (m^3/km^2 p.a.)	(10^6m^3 p.a.)
1953-1963	10	504.72	1794	50.47
1953-1972	19	322.79	600	16.96
1953-1978	25	418.81	594	16.73
1972-1978	6	96.02	566	16.02

Soil conservation methods such as contour bunding and afforestation have been introduced. It was estimated that an area of 4571 km^2, consisting of erodible soils (especially the black cotton soils) present in patches in the catchment area, was the most vulnerable and required urgent treatment. By 1978, an area of 3075 km^2 had been treated, 2085 km^2 by soil conservation methods and 233 km^2 by afforestation.

(2) The Guanting Reservoir

The Guanting Reservoir, built in 1956 on the Yongding River in north China, drains a catchment area at the dam site of 43,400 km^2. The average modulus of erosion of the watershed is about 3,000 t/km^2 p.a. with a maximum value of 18,000 t/km^2 p.a. The storage capacity of the reservoir is 2.29 X 10^9m^3. The annual runoff at Guanting station is 1.4 X 10^9m^3 and the annual sediment load is 81 X 10^6 t.

During the early years (1956-60) of the reservoir impounding, sediment deposition amounted to 360 X 10^6m^3. Since 1958, about 300 reservoirs have been built with total storage capacity of 1.5 X 10^9m^3 in the upstream reaches of the Guanting Reservoir on the main stream of the Yongding River. The largest is the Cetian Reservoir on the Sanggan River, whose catchment area of 16,700 km^2, controls 38 percent of the drainage area of the Guanting Reservoir. The soil conservation measures the warping with turbid flow at hyperconcentration (diverting heavy silt-laden flow, with high content of fertilizer, into farmlands for irrigation) resulted in a reduction of both runoff and sediment yield, as shown in Table 4.2.

Table 4.2. Reduction of runoff and sediment yield in Guanting Reservoir

Period	Annual precipitation (mm)	Precipitation in flood season (mm)	Annual runoff (10 m^3)	Annual silt discharge (10^6 t)
1951-1960	444	338	1863	79.54
1961-1970	410	313	1266	19.33
1971-1980	427	333		13.00

Consequently, a remarkable reduction in the amount of sedimentation and the rate of silting occurs (Table 4.3).

(3) Eel River (Serr, 1971)

The Eel River of California has a drainage area of about 9,330 km^2 and average annual natural runoff of about 8,360 X 10^6 m^3.

Table 4.3. Reduction of the rate of sedimentation in Guanting Reservoir

Period	Total amount of deposition ($10^6 m^3$)	Rate of siltation ($10^6 m^3$ p.a.)
1956-1960	350	70
1961-1970	82	8.2
1971-1980	73	6.6

The lower Eel River is one of the muddiest rivers in the world. The USGS has estimated its long-term average suspended sediment load at about 2,860 tons per km^2 of watershed annually. During the two-week flood of December, 1964, over 100,000,000 tons of sediment were carried by the Eel River.

Studies on the Eel River Basin were made to determine the sources and causes of the high sediment yields. The total sediment yield was divided into four sources: streambanks, landslides, sheet and gully erosion, and roads. The basin was divided into the five subbasins. Table 4.4 summarizes the annual sediment yield estimated for the Eel River Basin by principal source and subbasin.

Table 4.4. Sediment yield of the Eel River Basin ($10^6 m^3$ p.a.)

Subbasin	Streambank erosion	Landslides	Sheet and gully erosion	Road erosion	Totals
Outlet Creek-Pillsbury	0.57	0.23	0.44	0.023	1.27
Middle Fork	1.27	0.45	0.28	0.02	2.02
South Fork	0.88	0.77	0.22	0.018	1.89
Van Duzen	0.93	0.58	0.13	0.007	1.65
Main Eel	6.17	1.87	0.37	0.014	8.39
Totals	9.83	3.91	1.45	0.082	15.27
Percent	64.4	25.6	9.5	0.5	100

The USDA estimate of the sediment yield of the Eel River Basin amounts to almost 1630 m^3/km^2 p.a. This is the highest rate known for a basin of comparable size in the United States of America.

Land treatment measures recommended to reduce the sediment yield fell into two categories: remedial measures and management guidelines. Remedial measures were considered for each of the four sediment sources. The only remedial measures that were economically justifiable were to reduce sheet and gully erosion on privately owned grasslands used for grazing. Seeding and fertilizing of 1,085 km^2 of natural grasslands was recommended in order to increase the vegetal cover and reduce sediment yield. Reafforestation of 311 km^2 of harvested timberland was recommended.

The remedial measures considered to reduce streambank erosion were crib-dam drop structures and rock riprap bank protection; to stabilize landslides they included concrete crip restraining structures and a combination of rock riprap and horizontal drains; and to reduce road erosion, culvert installation and improvement and stabilization of road fills, cuts, and gutters by various means were suggested. None of these was found economically justifiable for basin-wide or subbasin-wide application.

With the land treatment program, the future sediment yield in the basin was expected to be about 20 percent less than without the program.

(4) Experiments on a small gully watershed

Empirical approaches to predicting the sediment yield for the sheet and rill erosion have provided many formulas, such as the Universal Soil Loss Equation formula, the SDR formula, etc. However, few studies have been carried out for the prediction of gully erosion and river bank erosion. In arid and semi-arid regions, gully erosion seems to be an important source of sediment yield.

Experiments on a small gully watershed as a unit have been made with various control measures to reduce the soil loss (Gong and Jiang, 1979). The following conditions were examined during the experiments:

a. Terraced fields

b. Afforestation

c. Grassland (growing alfalfa and melilot)

d. Check dam

e. Diversion of flood water for warping (use of sediment to fertilize soil) and irrigation.

Based on the statistical data observed in experimental areas of Chiuyan Gully and Hsintian Gully in the Northwest of China, Figure 4.1 shows the effect of controlling water and silt for each rainstorm by terraced fields, afforestation (locust trees with tree-age above four years, canopy density greater than 60 percent) and grassland vegetation. In Figure 4.1, the ordinates are the depth of runoff (R) and the soil loss (S') on the same type of sloping farmland not yet under control. The abscissae are the decreased depth of runoff (ΔR) and the weight of soil retained ($\Delta S'$) on the lands under various measures.

The effects of a check dam on reducing sediment transport are, first, to trap the sediments and diminish the sediment loads flowing out of the gully mouth; and second, to raise the bed of the gully channels and flatten the gully bottom, so that various types of erosion are weakened or checked.

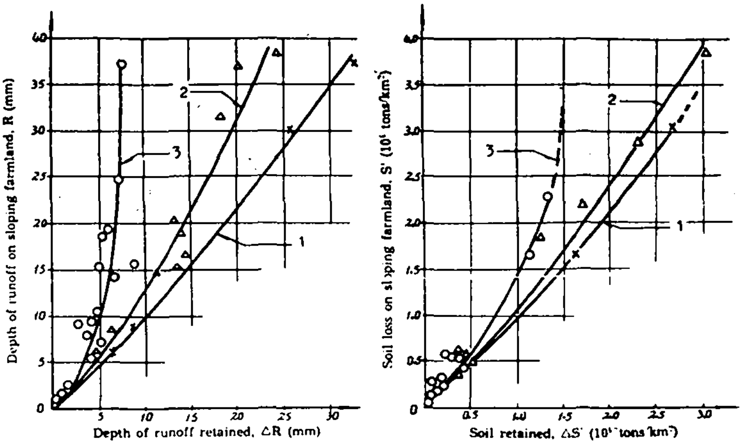

Figure 4.1. Effects of storing runoff and sediment by slope control (After Gong et al., 1979)

1. Terraced field: Horizontal surface with borders.
2. Forest: Tree age over four years, canopy density > 60%.
3. Grassed land: Alfalfa and melilot.

The sediment reduction in the Chiuyuan Gully is shown in Figure 4.2. The Chiuyuan Gully drains a catchment area of 70.1 km^2. After several years of erosion control work, the terraced fields, forests and grass-covered lands amounted to 24.1 km^2, comprising 34.4 percent of the catchment area. Three hundred and eighteen check dams were built. According to observations over a period of 18 years, the annual sediment yields at the gully mouths had decreased an average of 55 percent from those before erosion control was started. Some 9 percent was a result of slope control and 46 percent from check dam control. The capacity of slope control to hold the sediment has increased steadily with the enlargement of the control area, and has reached over 20 percent in recent years. However, the capacity of check dams has varied greatly with the increased number of years in service. The storage capacity was greater during the early years when the dams retained more silt; in subsequent years, the effects gradually diminished, but would increase again if the existing dams were raised or if new check dams were built. The effects were greater in dry years than in wet years.

A photo of terraced fields in Wupu Country, China, is shown in Figure 4.3.

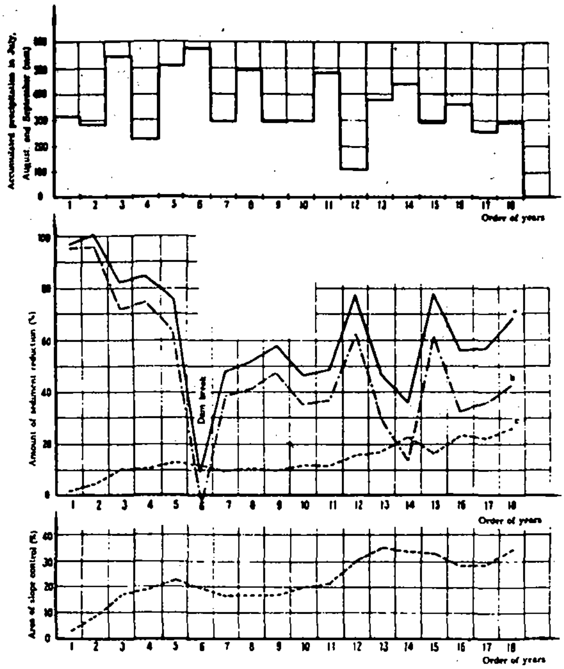

Figure 4.2. Effects of erosion control in successive years for Chinyuan Gully watershed (After Gong et al., 1979)

a. Total effect of sediment reduction.
b. Effect by gully control.
c. Effect by slope control.

Figure 4.3. Terraced fields in Wupu County, Shaanxi Province, China. Photo Ministry of Water Conservancy and Electric Power.

4.1.2 Trapping and Retention of Sediment by a Vegetative Screen

A vegetative screen is most effective in preventing the sediment from entering the reservoir or lake. Such screens whether artificial or natural at the head of the reservoir serve to diffuse the incoming flow, reduce its velocity and cause the sediment to deposit. Thus, a great amount of incoming sediment can be trapped at the head of the reservoir and prevented from penetrating farther into the basin.

It should be pointed out that the sediment deposited in an area of vegetation of the flood plain of a delta raises the same problems as progressive deposition on the delta after the dam completion. Farm lands in such sites may be injured through the rising watertable, and levees for flood protection might need to be constructed or heightened as a result of further deposition of sediment, if there are towns or industrial sites existing upstream of the reservoir.

The growth of tamarisk (salt cedar) along the Pecos River above Lake McMillan, USA, resulted in reduction of sediment deposition after 1915 (Stevens, 1936). The progressive storage capacity loss by silting from the 110.7 X $10^6 m^3$ capacity reservoir is seen in Figure 4.4. The significant feature of the curve is the decided flattening after 1915.

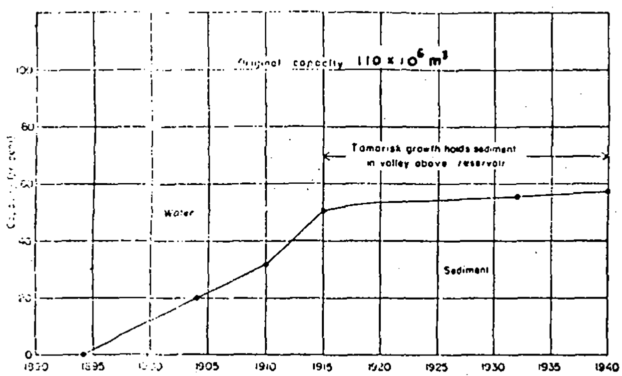

Figure 4.4 Curve showing silting in Lake McMillan, USA. Flattening of the curve after 1915 is attributed to the effect to tamarish growth in the valley above the head of the reservoir (After Stevens, 1936)

The second example is the Elephant Butte Reservoir built on the Rio Grande, USA. Before 1930, relatively few salt cedar plants were known to inhabit the upstream areas (Lara, 1960). Infestation of salt cedar in the Middle Rio Grande Valley increased significantly in both area and density between 1935 and 1947. In the reach of the Rio Grande between Bernardo Bridge and San Marcial, a further 12.1 X $10^6 m^2$ became colonized during the 1947 to 1955 period.

These trees decrease the stream currents and screen out a considerable portion of the sediment which is now being deposited above the crest elevation of the dam.

The following table shows data on both the above crest and below crest deposits for comparison:

Table 4.5. Deposition of sediment at the head of reservoir due to vegetative screen, Elephant Butte Reservoir

Period	Above crest sediment deposits ($10^6 m^3$)	Below crest sediment deposits ($10^6 m^3$)	Total sediments ($10^6 m^3$)	Percentage of sediment deposited above dam crest
1951-1935	7	450	482	1.57
1935-1947	27	90	116	22.91

The effects of vegetation have been described in a sedimentation survey document (Lara, 1960). Salt cedar infestation is an important factor in the aggradation of the channel and flood plain of the Rio Grande. Its presence along the channel of the stream accelerates the natural levee-building process by causing a decrease of the stream velocity next to the banks. When overflow occurs, coarser sediments are deposited in the vegetated areas. In this manner, the rate of natural levee building is increased and the slope of the natural levee away from the channel is steeper than it would be without the influence of vegetation. Vegetation causes immediate deposition of sediments adjacent to the stream and lessens the amount that will be deposited on the flood plain some distance from the stream.

Avulsion can break through the levee, which is strengthened by the presence of vegetation; the size of the avulsion is larger than it would be otherwise. Downstream from the avulsion, the river is effectively prevented from reentering its old channel and is held to a new course behind the natural levee. The presence of vegetation in the flood plain accelerated the deposition of sediment, and if the flood plain has a dense vegetative cover when the avulsion occurs, practically all of the sediment load of the stream will be deposited in this area.

The building of a natural levee and aggradation of the stream channel is frequently the cause of swamp or boggy areas that occur on the adjacent flood plain. These areas are condusive to a further growth of vegetation. If the vegetation is luxuriant, a sediment plug may be formed which splits the stream into many channels. This, in turn, encourages deposition and further increases the size of the sediment plug. The plug generally forms when there is a large flood and an avulsion occurs at a point where the stream channel is several feet above its flood plain. When the stream is split into different channels, it is unable to scour any one particular channel to convey the water through this reach.

One ill-effect of increased vegetation cover is the increase of water consumption. According to Maddock (1948) the increased vegetation in the upstream reaches led to over 123 X $10^6 m^3$ of water being consumed annually by transportation (Maddock, 1948). This loss is almost 10 percent of the yearly water supply (Garde et al., 1978).

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The third example is the Hongshan Reservoir on the Loaha River, a tributary of Xiliao River in northeast China. This multipurpose reservoir has a storage capacity of 2.5 X $10^9 m^3$ and receives an annual runoff of 918 X $10^6 m^3$. The annual sediment discharge is 43 X 10^6 tons, with an average sediment concentration of 46.8 kg/m^3, and the median diameter of 0.02 mm during the flood season.

The reservoir began operating in 1960. Up to 1977, a total of 475 X $10^6 m^3$ sediments had been deposited. Between 1966 and 1977, 95 percent of the inflowing sediment had been deposited upstream of Range 13 (15 km from the dam site), i.e., on the flood plain at the head of the reservoir where a vegetative screen was formed of the plants *Scirpus yagara*, *Typha latifolia*, etc. which grow 2 to 3 m in height and cover an area more than 4 km wide, and 15 km long (Zhao, 1980). The longitudinal profile of the flood plain is shown in Figure 4.5 and the varying proportion of sediment deposited in the upstream reach of Range 13 is shown in Figure 4.6.

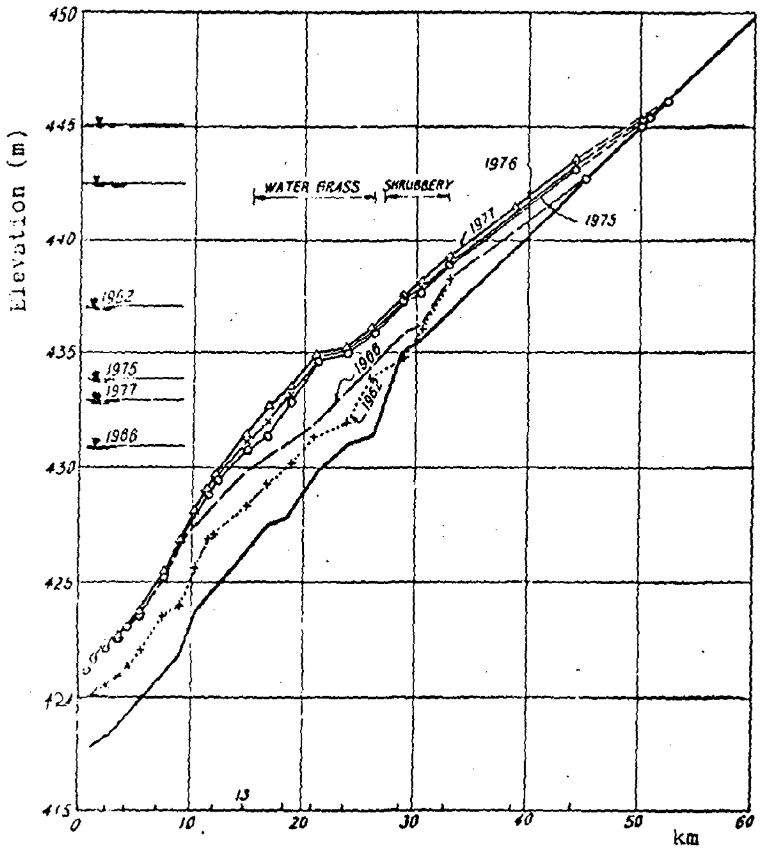


Figure 4.5. Flood plain of Hongshan Reservoir (After Zhao, 1980)

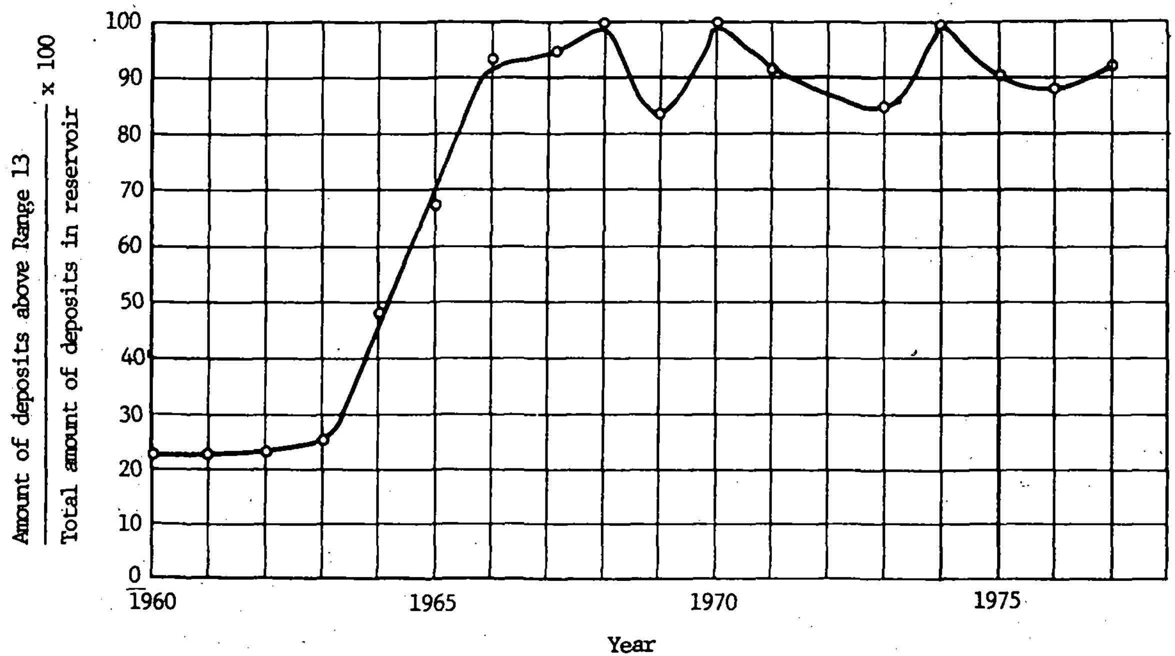


Figure 4.6. Ratio of amount of deposits above Range 13 to the total amount of deposits in Hongshan Reservoir

4.1.3 Bypassing of Heavily Sediment-Laden Flows

The construction of bypassing channels or conduits is one of the principal methods used to control the inflow of sediment to impounding reservoirs. Generally, a great amount of sediment is carried by the river flow during the flood periods, especially in the arid and semiarid regions. Thus, when a large part of the flow with high concentration is bypassed through a channel or tunnel, serious silting in the reservoir may be avoided. Several schemes are described below.

(1) The Hushan Reservoir was built for the purpose of irrigation in 1969 on a tributary of Hutuo River in north China. The storage capacity of the reservoir is 4.23 X $10^6 m^3$. The annual runoff is estimated to be 5.91 X $10^6 m^3$ and the annual sediment load of inflow, 200 X $10^3 m^3$.

In 1976, measurements were made showing that a volume of 2.26 X $10^6 m^3$ of sediment had been deposited in the reservoir; i.e., about 54 percent of the total capacity was silted up. The annual rate of siltation for the period from 1959 to 1976 was 130 X $10^3 m^3$ p.a.

In order to reduce the siltation in the reservoir, a supplementary project was adopted. This included the construction of a small dam for flood detention at the entrance of the Hushan Reservoir near the end of the backwater reach, and a diversion canal for diverting the heavily sediment-laden flows downstream of the dam during flood seasons. The diverted flows containing high concentrations are used for warping and irrigation.

A substantial reduction of siltation resulted from this work. The total sediment deposition between 1976-1981 was 50 X $10^3 m^3$ from field measurement. The rate of silting was some 10 X $10^3 m^3$ p.a., about one thirteenth of the annual average before 1976.

(2) The Tedzen Reservoir was built in 1950 on the Tedzen River in the Turkmentistan in the USSR, for the purposes of irrigation and flood protection for the cities of Tedzen and Kirofsk, as well as a railway line.

The River Tedzen has a length of 1124 km with an annual runoff of 700 X $10^6 m^3$ and an annual sediment discharge of 8.2 X $10^6 m^3$. The average measured sediment concentration in the flood season was 16-20 kg/m^3, the maximum sediment concentration observed being 94 kg/m^3. During the floods more than 50 percent of the sediment is finer than 0.01 mm.

The original storage capacity of the reservoir was 177.2 X $10^6 m^3$. In 1962, a sedimentation survey showed that a volume of 79 X $10^6 m^3$ had become silted up, i.e., 44 percent of the total initial storage capacity of the reservoir had been lost. The rate of siltation for the period 1950-1962 was 6.5 X $10^6 m^3$ p.a.

The very high rate of progressive siltation coupled with the rapid decrease of the storage capacity of the Tedzen reservoir began to threaten the security of the cities and the railway from flooding. A flood diversion canal was constructed from the upstream reach (Figure 4.7), passing the side of the Hayz-Hansk Reservoir and crossing under the railway line by a culvert. An average of 28 percent of the flood waters bearing heavy sediment loads are diverted away. Some enter the Hayz-Hansk Reservoir, but more are discharges downstream, resulting in an annual average reduction of sediment deposition by about 2.3 X $10^6 m^3$ (Hachaturian et al., 1966)

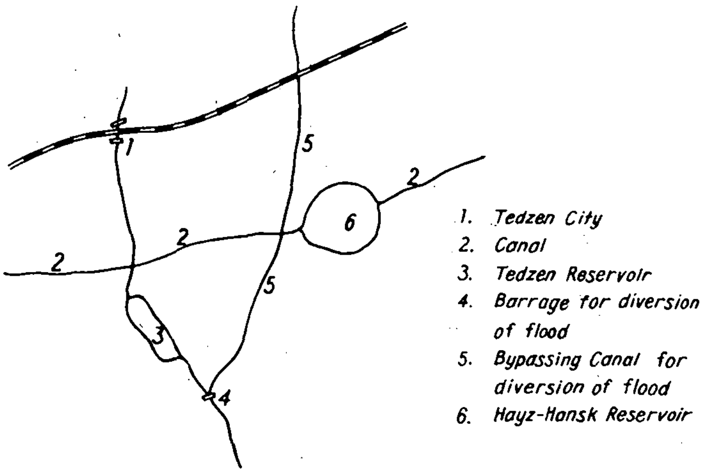


Figure 4.7. Sketch map of Tedzen Reservoir showing bypass canal

(3) <u>In Switzerland</u>, the diversion of silt-laden floods from the head of reservoirs through bypass galleries directly to the stream below the dam has been examined for several reservoirs (Swiss National Committee on Large Dams, 1982).

A bypass gallery was added in 1974 at the head of the Palagnedra Reservoir to divert sediment-laden floods directly downstream. The diversion gallery, 1,760 m long has a section of 30 m^2 and a slope of 2 percent with a freeflow capacity of 225 $m^3.s^{-1}$. The arrangement is depicted in Figure 4.8.

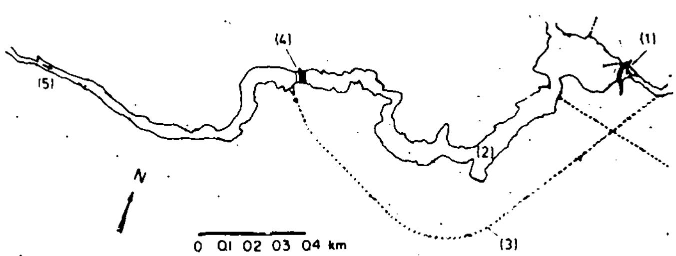


Figure 4.8 Sketch map of Palagnedra Reservoir showing bypass gallery (After Swiss National Committee on Large Dams, 1982)
(1) Dam, (2) Reservoir, (3) bypass gallery for sediment laden floods, (4) Melezza River

A planned diversion gallery for the Gebidem Reservoir was abandoned due to cost.

(4) A similar scheme in Switzerland was the diversion dam and bypass tunnel constructed above the Amsteg Reservoir on the Reuss River which was completed in 1922 for power production. Reed (1931) described them in detail as follows:

> "The Amsteg Dam is 32 m high, with a storage capacity of only 197 X $10^3 m^3$. The drainage area above the reservoir is 404 km^2, and the flow of the Reuss River ranges from a minimum of 1.98 $m^3.s^{-1}$ to a flood flow of 396 $m^3.s^{-1}$. In order to prevent river debris from filling the reservoir, a bypass tunnel was constructed around the reservoir (Figure 4.9). The tunnel is 305 m long and has a cross-sectional area of 20.9 m^2 and a flow capacity of 221 $m^3.s^{-1}$. The flow through the tunnel is regulated by gates, so that the flow into the pond is only sufficient to supply the power-plant demand. During the low-water season, when the full flow is utilized through the plant, little sediment is carried by the stream. During flood periods about one-third of the total flow is spilled over the diversion dam and passes through the pond. This is the part of the stream flow near the surface, which carries only silt and fine sand, while the large part of the flow heavy with sediment is bypassed through the tunnel. Observations made after three years of operation showed that deposition in the pond had been insignificant."

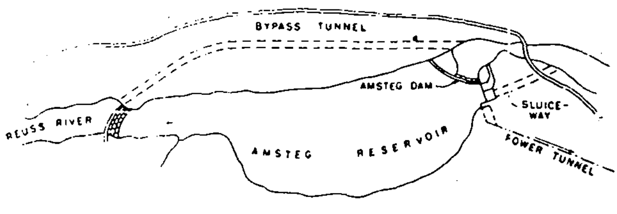


Figure 4.9. Sketch map of Amsteg Reservoir showing bypass tunnel (After Reed, 1931)

The construction of a tunnel around a reservoir is very expensive. The presence of suitable topographic features near a reservoir which permit the installation of a short bypass canal or conduit is invaluable economically.

It seems that, to prevent silting for power dams which have small reservoir capacity, and for which it is necessary to maintain a constant head, it is often economically feasible to build bypass structures. Other benefits include the reduction of sediment damage to water turbines and other power equipment.

4.2 METHOD OF MAXIMIZING SEDIMENT THROUGH FLOW

4.2.1 Flow Regulation During Floods

The purpose of regulating the flow by a reservoir during the flood season is to release as much sediment as possible from the reservoir by making use of the silt carrying capacity of the flood. Generally, the regulation of flow is achieved by lowering the water level during the flood season by operating the deep or bottom outlets under controlled (partial opening) or uncontrolled conditions.

During the period of rising water level of a flood in a detention reservoir, the outflow silt discharge is always smaller than that of the inflow, as a result of the backwater effect, and the consequent decrease in the velocity of the flood waters. Subsequently, during the lowering of the water level at the dam in the absence of a backwater effect, the outflow silt discharge is often greater than the inflow, due to erosion occurring in the reservoir.

In the following paragraphs the characteristic features of flood flushing in some reservoirs, and the flow features in detention reservoirs will be discussed. We hope that this discussion will lead to a more profound understanding of their physical properties.

A. Controlled Flow

(1) Heisonglin Reservoir (Zhang et al., 1976)

Operation of reservoirs by lowering the pool levels during the flood season to sluice out waters with high sediment concentration is normal in many reservoirs in China. The mode of operation is based on the fact that 80 to 90 percent of the annual sediment load carried by the river is discharged in July and August, whereas only 25 to 50 percent of the annual runoff occurs in the same period. This is a characteristic feature of the arid region of northern China which is underlain by loess deposits.

The Heisonglin Reservoir is a small reservoir on a tributary of the Yellow River in northwestern China. When the 45.5 m-high dam was constructed in 1959, it had an initial storage capacity of 8.6 X $10^6 m^3$, which had become reduced to a capacity of 5.87 X $10^6 m^3$ by 1973. The annual runoff is 14,200,000 m^3. Before July 1962, the reservoir was operated as an impounding reservoir, resulting in serious siltation. The average rate of sediment deposition amounted to 540,000 m^3 p.a. After August 1962, when the mode of operation of the reservoir was changed to involve lowering the pool level during flood season and impounding water outside the flood season, the rate of sediment deposition decreased to 93,000 m^3 p.a.

The reservoir is equipped with a bottom outlet having a discharge capacity of 10 m^3s^{-1}.

From the hydrographs of a flood period for the Heisonglin Reservoir (Figure 4.10), it is apparent that the discharge of silt from the reservoir was maintained for more than 24 hours, while the duration of the silt inflow peak was about 5-6 hours. The inflow with high concentration was detained in the reservoir, forming the muddy pond before the dam, and the fine sediment settled very slowly in a medium of high concentration. As a result, the sediment concentrations discharged from the outflow were higher, ranging from 750-150 kg/m^3.

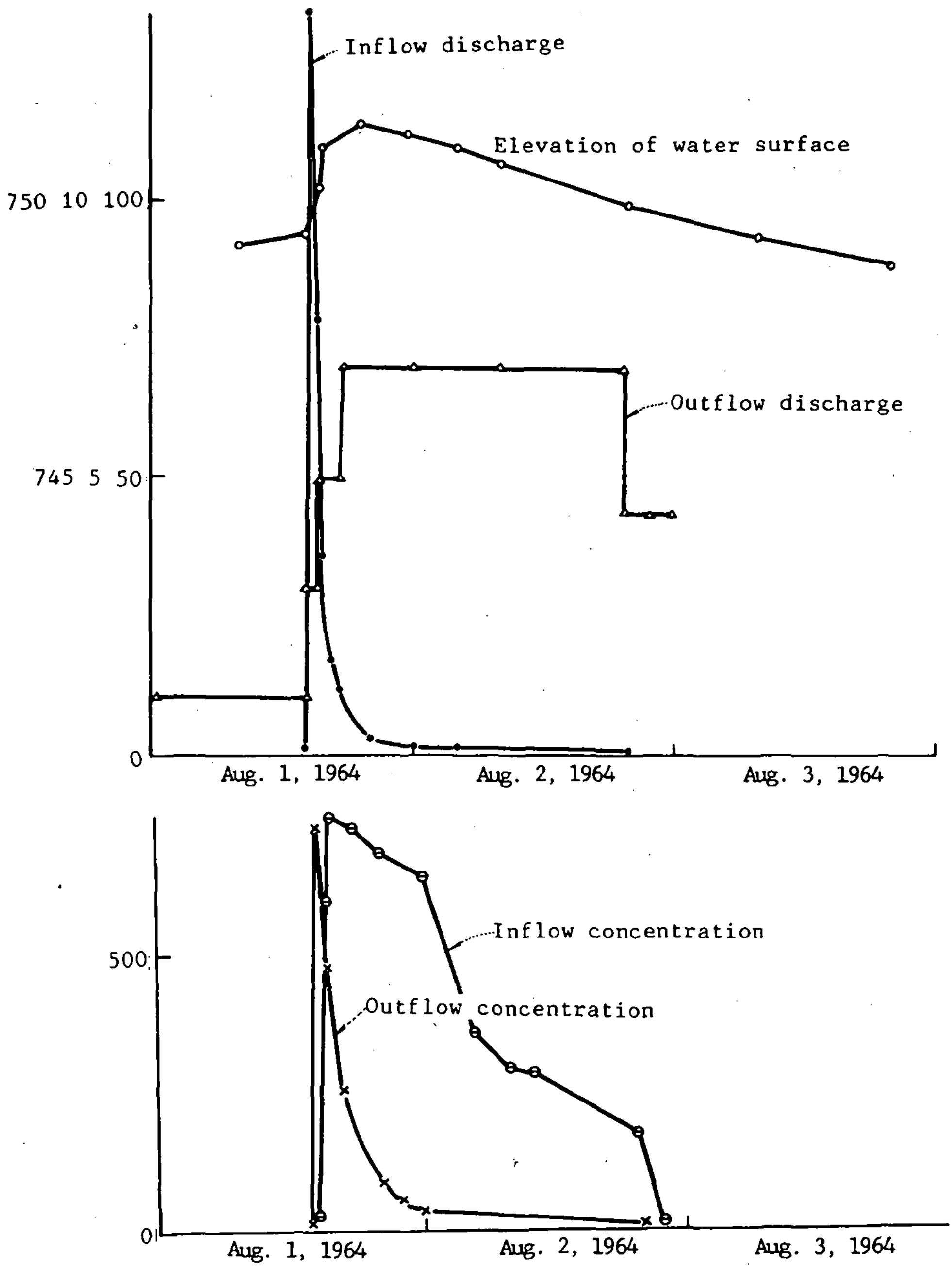


Figure 4.10. Hydrographs of a Flood in Heisonglin Reservoir

This mode of operation of storing the clear water during the nonflood season, discharging the muddy water during the flood season, and diverting the muddy water for irrigation and warping not only decreases the rate of silting, thereby preserving the long-term usable storage capacity of a reservoir; but, it also enables both the sediment and flood water to be used in agricultural production.

(2) Honglingjin Reservoir

The medium sized Honglingjin Reservoir in north China is operated in a similar way. The dam is 42 m high and the reservoir behind it has a storage capacity of 16,600,000 m^3. The annual runoff is 43,200,000 m^3. Flow is regulated by lowering the pool level during the flood season, as shown in Figure 4.11. In the water year under examination, between June 15 and the end of August, 1974, the water level was lowered for sluicing deposited sediment from the reservoir.

(3) Sanmenxia Reservoir

The third example is from a large reservoir. The flood flushing operation was adopted in the Sanmenxia Reservoir about 10 years after the original completion of Sanmen Gorge Dam (Zhang and Long, 1981).

During flood flushing, the regime of sediment movement was in the form of open-channel flow or density currents. The ratio of outflow to inflow of sediment in the former case was usually greater than that in the latter case.

The Sanmenxia Reservoir is located in the middle reaches of the Yellow River. The Sanmen Gorge dam was completed in 1960 as part of a multipurpose project. The average annual runoff was 43.2 X $10^9 m^3$, of which 60 percent reached the reservoir during the flood season from July to October. The annual sediment discharge was 1.6 X 10^9 tons, the yearly average suspension concentration was 37.8 kg/m^3, and the maximum concentration measured reached 933 kg/m^3.

The dam site is located in a gorge 114 km downstream of Tongguan, which is at the confluence of the Yellow River and the Wei Ho River. Tongguan serves hydraulically as a control section, because the bed width at this cross section is only 1 km, whereas the width in the confluence zone is more than 10 km. The stage at this confluence point is considered as a local base of erosion. The stage at the control section, also affects the flow pattern and the sediment movement in the upstream reaches of both the Yellow and Wei Ho Rivers above Tongguan (Figure 4.12).

The history of operation of the Sanmenxia Reservoir may be considered in 5 stages (Table 4.6).

After the dam was completed, during the first six months the reservoir was operated by storing water, and the stage at the control section Tongguan was raised due to the filling of the reservoir and sediment deposition, which resulted from the backwater effect. Then deposition began extending farther and farther upstream. On the Wei Ho River backwater, deposition extended upstream about 250 km above the dam, that is, 136 km upstream from Tongguan. This deposition of sediment led to a rise of the water level in the Wei Ho River and the watertable level in this region was also raised. These effects made flood control and agriculture more difficult in the region.

Serious siltation occurred in the reservoir from 1960-62, because the flushing capacity of the outlet works was limited. In the 18-month period about 1.8 X 10^9 tons of sediment, i.e., 93 percent of the incoming silts were deposited

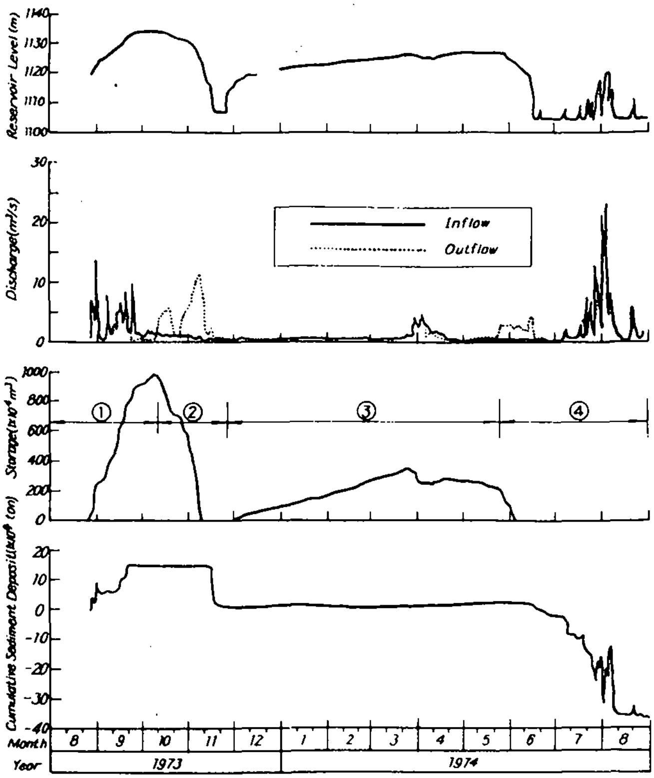


Figure 4.11. Coordinated hydrographs for Honglingjin Reservoir (After Zhang et al., 1976)

1. Storing water after flood season.
2. Irrigating the farmland after autumn.
3. Storing water in winter and spring.
4. Scouring the sediments during flood season.

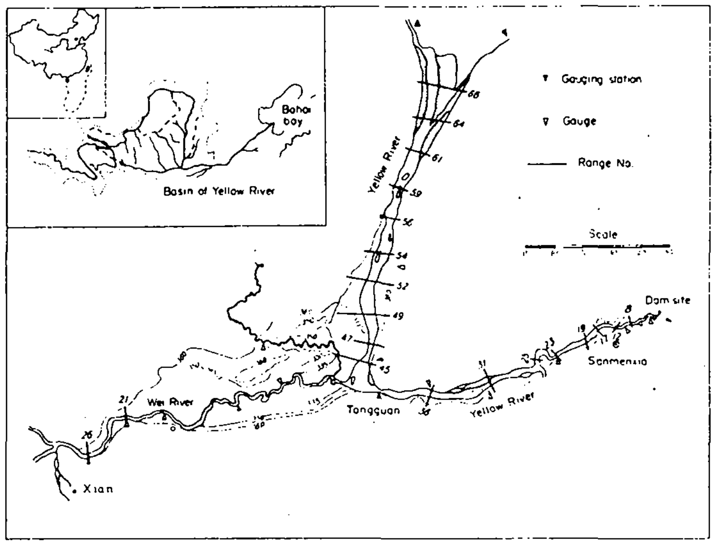


Figure 4.12. Plan of Sanmenxia Reservoir (After Long et al., 1981)

Table 4.6. Sediment outflow at different stages of operation of the Sanmenxia Reservoir

Stage	Period	Operation Scheme	Water level at dam site		Sediment quantity in outflow	
			max.	average	10^9T	$\frac{\text{outflow}}{\text{inflow}}$ %
1	Sept. 1960 - Mar. 1962	Storing water	332.58	324.04	0.11	6.8
2	April 1962 - July 1966	Flood detention and sediment sluicing by lowering the water level through 12 opening at 300 m	325.90	312.81	3.39	58.0
3	July 1966 - June 1970	Flushing by additional opening 2 tunnels and 4 penstocks	320.13	310.00	7.38	82.5
4	July 1970 - Oct. 1973	Flushing: 2 tunnels 4 penstocks 8 diversion outlets	313.31	298.03	5.93	105.
5	Nov. 1973 - Oct. 1978	ditto	317.18	305.60	6.68	100.

in the reservoir. At the same time the elevation of the river bed at Tongguan increased by 4.5 m.

It was quite apparent that if these conditions were to persist, the reservoir would be in danger from this sediment deposition. Consequently, the mode of operation had to be changed as a first step, and reconstruction of outlets was started.

After 1962, although the reservoir operation was changed by lowering the water level during the flood season, the deposition rate was still large, with a further 3.72 X 10^9m^3 of sediment being deposited before June 1966. This was the second stage of operation.

It was apparent that the sluicing capacity of the dam should be greatly improved in order to diminish the siltation within the reservoir, and also to lower the bed elevation at Tongguan in order to control the upstream extension of backwater deposition in the Wei Ho River. Better sluicing would regulate the outflow regimes, and decrease the aggradation in the Yellow River below the dam.

Since then several steps have been taken to improve the sluicing capacity of the dam (Figure 4.13). Two tunnels were dug along the left bank at an elevation of 290 m, and four of the eight penstocks were converted to sluiceways. Between 1970 and 1973, 8 of 12 diversion outlets of 280-m elevation, which were used during construction, were reopened.

During the third stage of operation, from July 1966 to June 1970, after tunnel excavation and penstock conversion, 82.5 percent of the incoming sediment was released by lowering the water level within the reservoir during the flood season. However, the bed elevation at the control section of the confluence, Tongguan, was still rising due to the backwater deposition.

In the fourth stage of operation, from July 1970 to October 1973, when the additional diversion outlets were reopened, the reservoir was operated for flood detention and sediment sluicing. The sediment outflow-inflow ratio reached 105 percent and the bed elevation at the Tongguan control section fell by nearly 2 m.

As a result of lowering the water level above the dam, the hydropower units achieved about 20 percent of the design output (which required regular high water level at 360 m).

In the fifth stage, since the outflow discharge capacity had been considerably increased; the reservoir could be operated to control the heavy floods and, by restraining the water level, minimize the backwater deposition at Tongguan; thereby, restricting further upstream deposition in the Wei Ho River. It could also serve for ice-jam control and provide water for irrigation and hydropower by impounding during nonflood seasons. The sediment outflow-inflow ratio was adjusted to about 100 percent, that is, to a yearly or long-term average balance of the sediment deposition and erosion, to preserve the useful storage capacity. Under such operational conditions, the sediments flushed from the reservoir during the flood season are carried by greater water discharges than were possible before reconstruction, as shown in Figure 4.14a and b.

Successive profiles of the floodplain and the thalweg in the Sanmenxia Reservoir, showing the depositional and erosional effects in the reservoir at different stages of operation are given in Figures 4.15 and 4.16.

In conclusion, the characteristics of flood flushing have been utilized to minimize the deposition within the Sanmenxia Reservoir to retain its long-term storage capacity and to increase the silt-carrying capacity of the lower reaches of the Yellow River.

B. Uncontrolled Flow

The flow and sediment features of the detention reservoir operated for flood control purposes are similar to those of reservoirs in which the outlets are opened for flushing the flood waters and sediment during the flood season.

In a dry detention reservoir, where the dam is only for flood reduction, surges are generally evolved. Sediment is deposited as a result of the backwater effect and surges occur in the reservoir when the water level is rising. When the water begins to draw down, the velocity of the flow is increased and erosion of the deposits occurs. This kind of erosion in the reservoir is referred to as retrogressive erosion, because erosion always progresses in the upstream direction during the sudden drawing down of the water level. During this process, much of the sediment previously deposited above the dam is eroded or a large proportion of the incoming sediment is carried through and released from the reservoir.

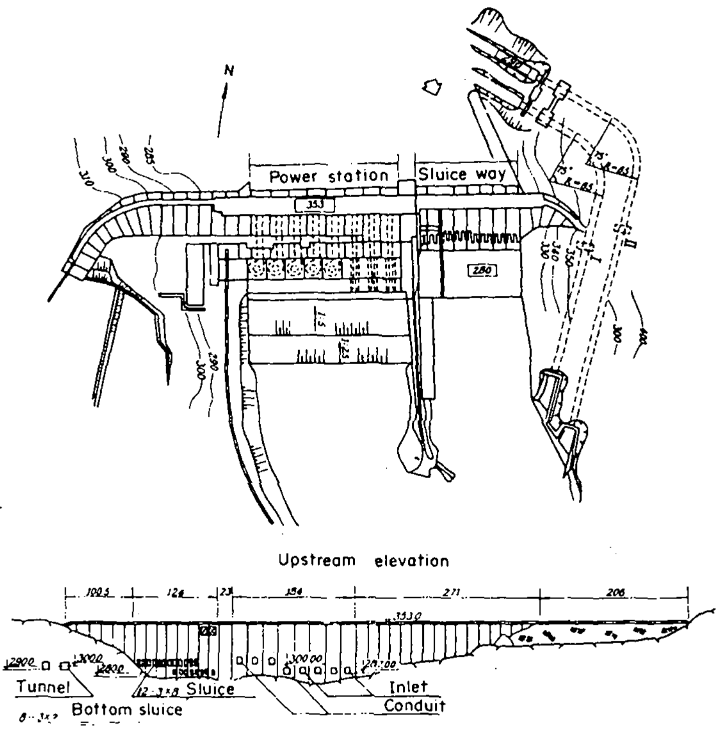


Figure 4.13. General plan for reconstruction of Sanmenxia Reservoir (After Long et al., 1981)

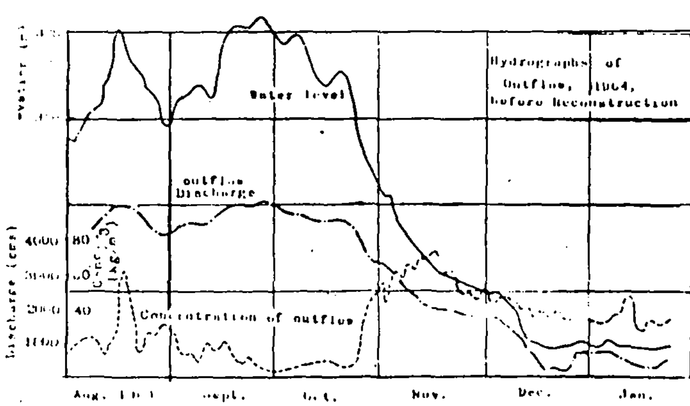
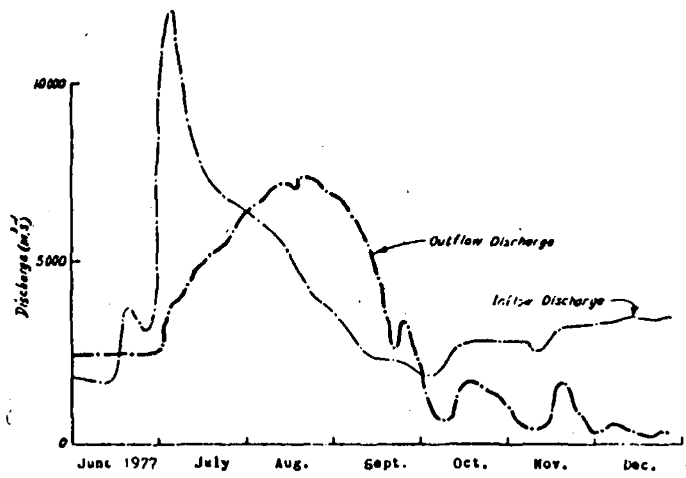


Figure 4.14a. Hydrographs of the outflow, 1964, before reconstruction, Sanmenxia Reservoir



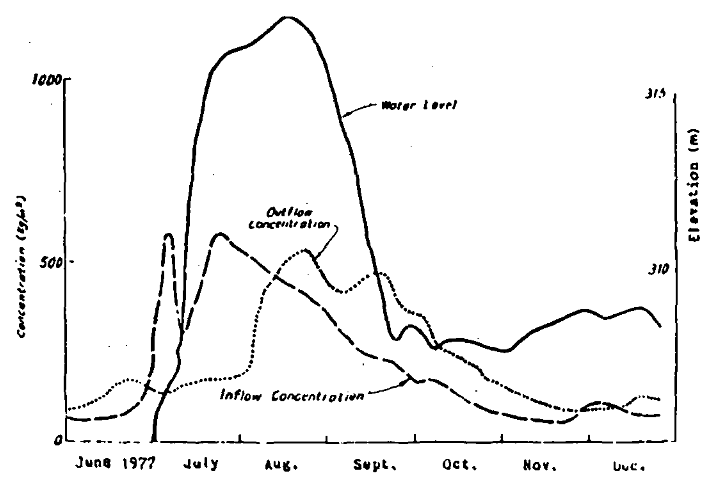


Figure 4.14b. Hydrographs of the inflow and outflow, 1977, after reconstruction, Sanmenxia Reservoir

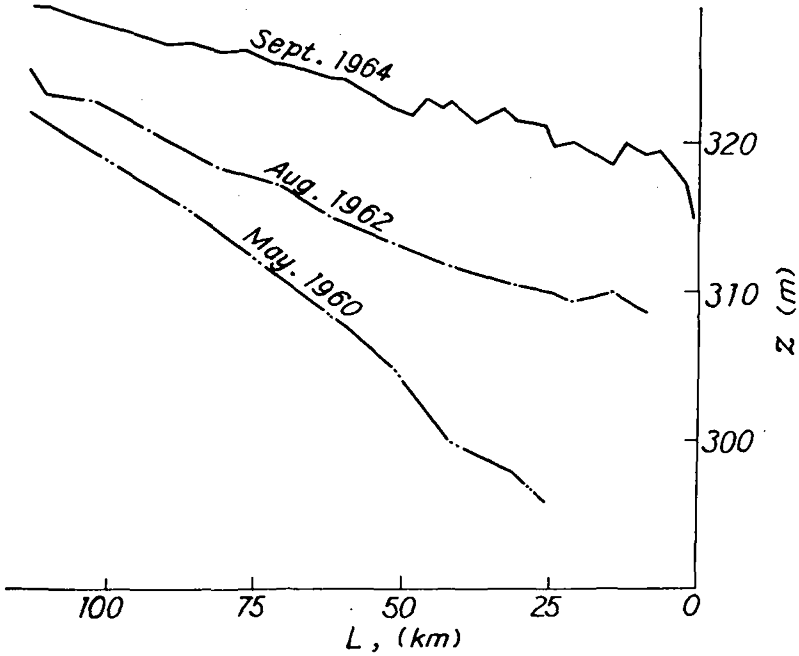


Figure 4.15. Profiles of flood plain in Sanmenxia Reservoir

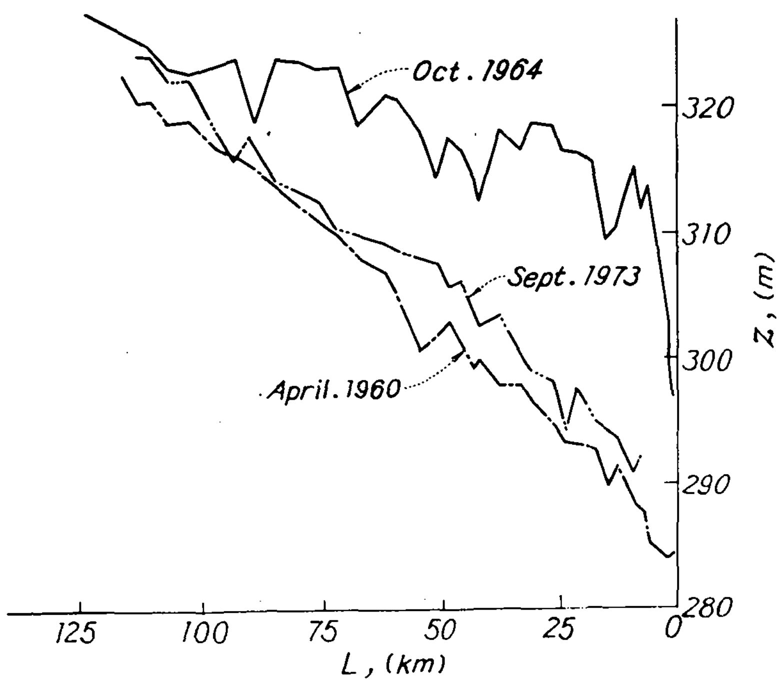


Figure 4.16. Profiles of thalwag in Sanmenxia Reservoir

(1) Sedimentation of the Germantown and Englewood Reservoirs

In an analysis of sedimentation in retaining basins in the Great Miami River Valley of southwestern Ohio, U.S.A., Curtis (1968) considered five dry basin reservoirs which retain water only during periods of excessive runoff. The reservoirs are formed by dams between 20 m and 34 m high, all of which have uncontrolled outlets with fixed openings. Water is held only during floods, and the retention time is usually only a few days. Under these operating conditions, the amount of sediment deposited is not significant. The Germantown and Englewood retaining basins were the only two with a measurable amount of deposited sediment. The sedimentation was ascribed to the relatively small ratio of outflow to inflow of flood discharges as compared with the other three retaining basins. The sediment deposition and the trap efficiency of the two silting reservoirs are shown in Table 4.7.

Table 4.7. Trap efficiency of the Germantown and Englewood retaining basins

	Germantown	Englewood
End of dam construction	1920	1920
Spillway elevation in m	249	274
Capacity at spillway elevation in $10^6 m^3$	130	384
Discharge through outlets with water level at spillway elevation in $m^3.s^{-1}$	283	340
Average annual sediment deposition, 1937-1939, in tons	34,800	64,400
Annual sediment production from watershed in tons	103,000	188,000
Trap efficiency, percent	33	34

(2) The Sanmenxia Reservoir During the Detention Period Before Dam Completion

It may be interesting to introduce some information about the deposition and erosion in the Sanmenxia Reservoir during the short period of retention in 1959. At this stage, 12 diversion outlets were used for releasing floodwater, during construction of the dam, when the backwater length was about 40 km. The hydrographs of discharge and silt concentration of inflow and outflow are depicted in Figure 4.17.

Repeated bed level measurements were carried out by the range method at different times during the year. The pattern of net deposition along the length of the reservoir between March and September 1959 is shown in Figure 4.18. this resulted from sediment settling when the water level was in the process of rising. During the period from September to November 1959, erosion occurred in most parts of the reservoir, as also plotted in Figure 4.18. By comparing the amount of deposition and erosion on each range, figures for the net deposition of sediment and the trap efficiency of the reservoir as a whole, may be estimated. Example cross sections on ranges 18 and 20 measured during the depositional and erosional phases are presented in Figure 4.19. In the wide cross section

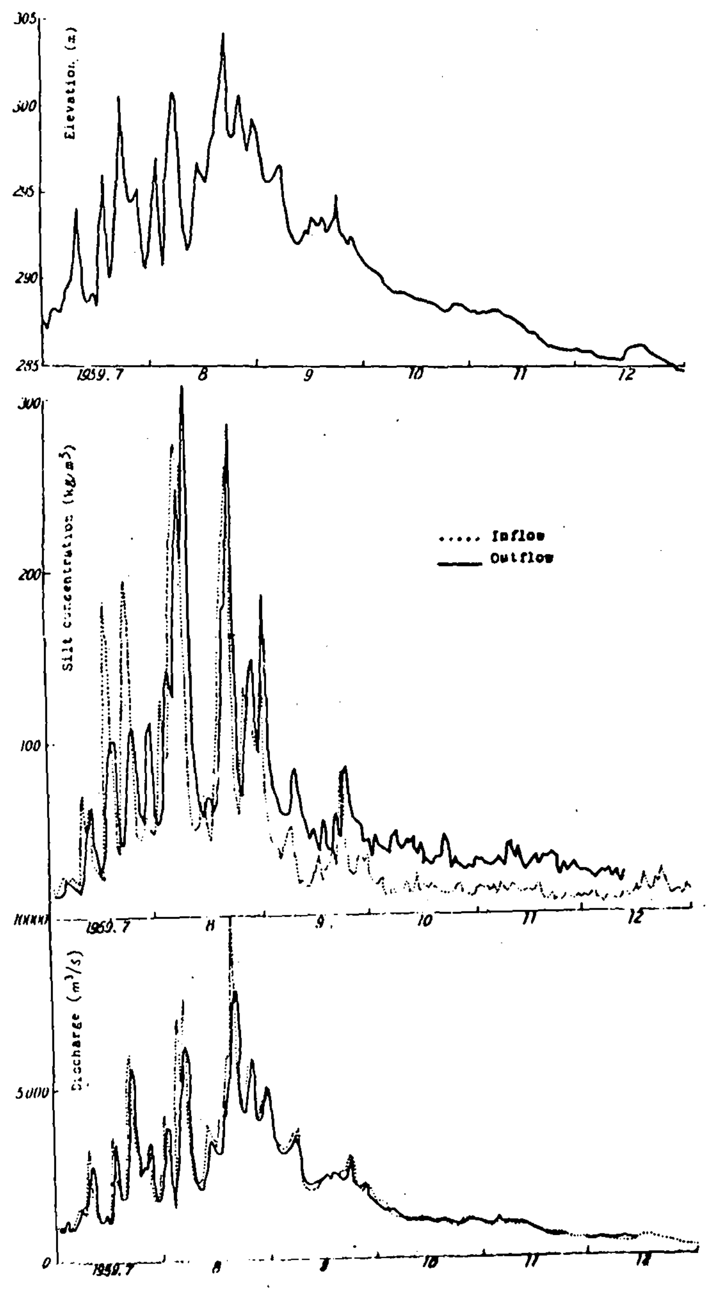


Figure 4.17. Elevation - sediment concentration - discharge, July to December 1959, Sanmenxia Reservoir

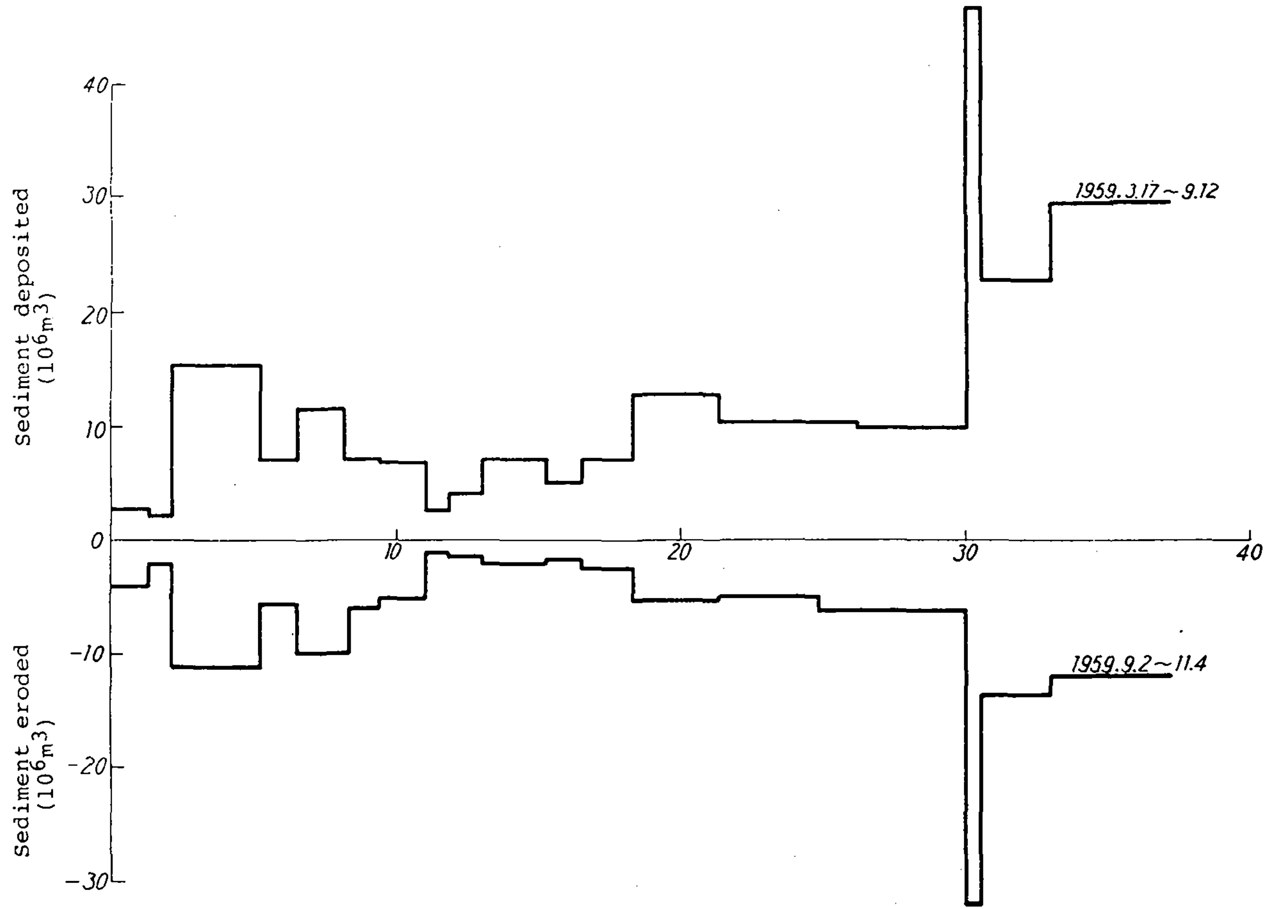


Figure 4.18. Erosion and deposition distribution along the length of Sanmenxia Reservoir, 1959

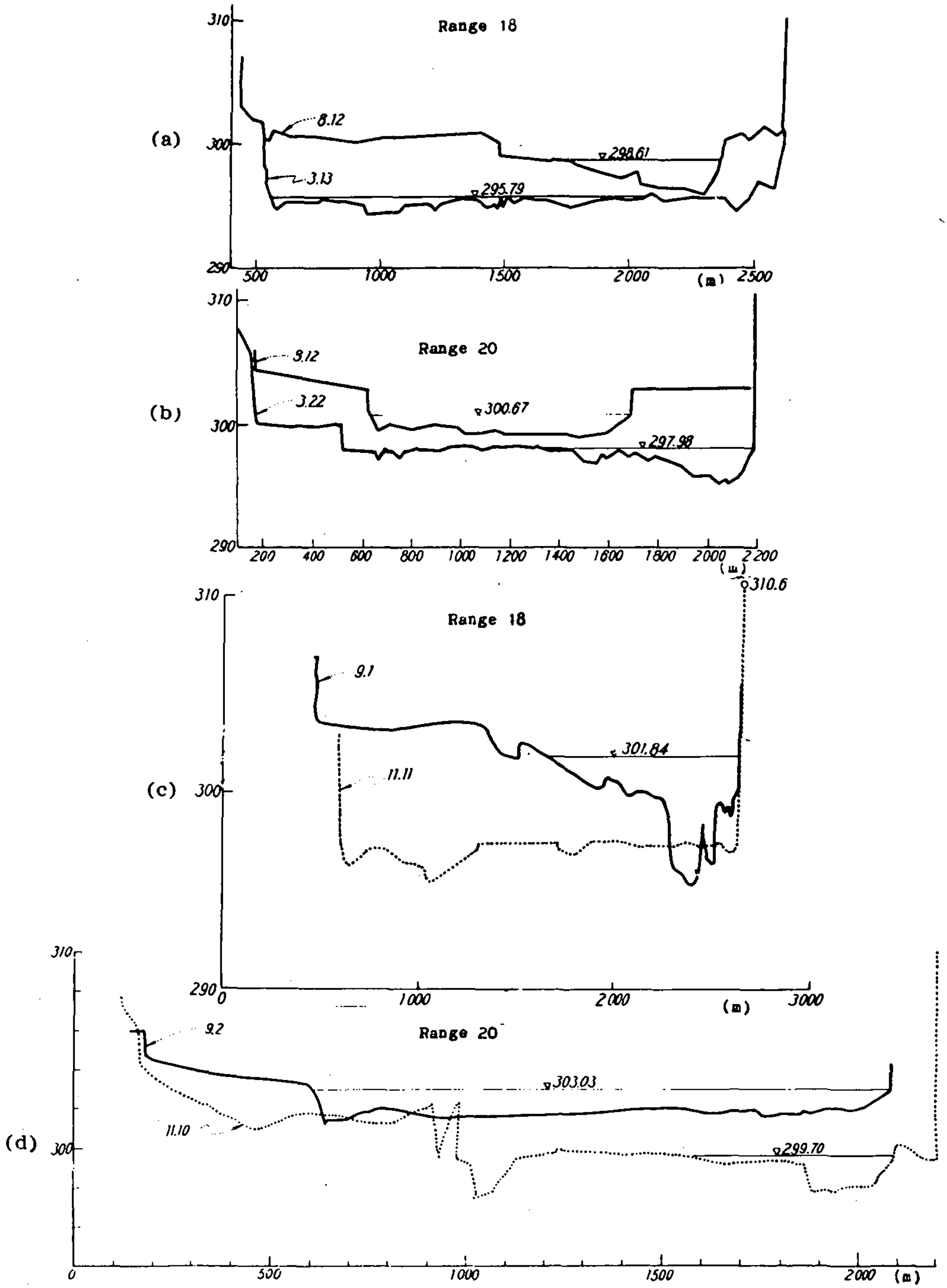


Figure 4.19. Deposition and erosion of Range 18 and 20 during detention period of Sanmenxia Reservoir, 1959

(Range 20), the flood plain, which formed during the silting period from March to August, could not be eroded during drawing down of the water level between September and November (Figure 4.19). The material forming the flood plain was entirely of sediment trapped during the 1959 flood period. However, in the narrower Range 18 virtually all of the deposited material was removed during draw down.

(3) Erosion in 1964, Sanmenxia Reservoir (gates were opened during the nonflood season)

Another example of retrogressive erosion occurred in 1964 in the Sanmenxia Reservoir. The variations of the water level within the reservoir are shown in Figure 4.20, in which the variation of silt concentration at different ranges are also plotted. The concentration hydrographs of the outflow released from the reservoir show higher suspension levels than those at Range 22 (42.3 km from the dam site) from where the values are higher than on Range 41 (113.5 km). The erosion rate appears to be greater within the area from 42.3 km to the dam site than that in the area from 42.3 to 113.5 km, showing that the stronger erosion occurred near the dam, which is one of the characteristics of retrogressive erosion. The same pattern is very clearly indicateed by the sequence of reservoir water level (floor) profiles given in Figure 4.21, in which erosion near the dam is much greater than that towards the head of the reservoir.

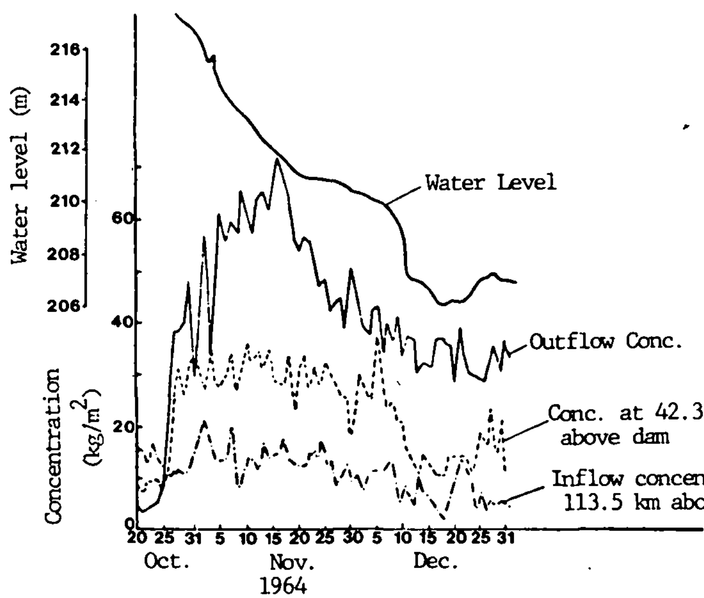


Figure 4.20. Retrogressive erosion in Sanmenxia Reservoir, 1964

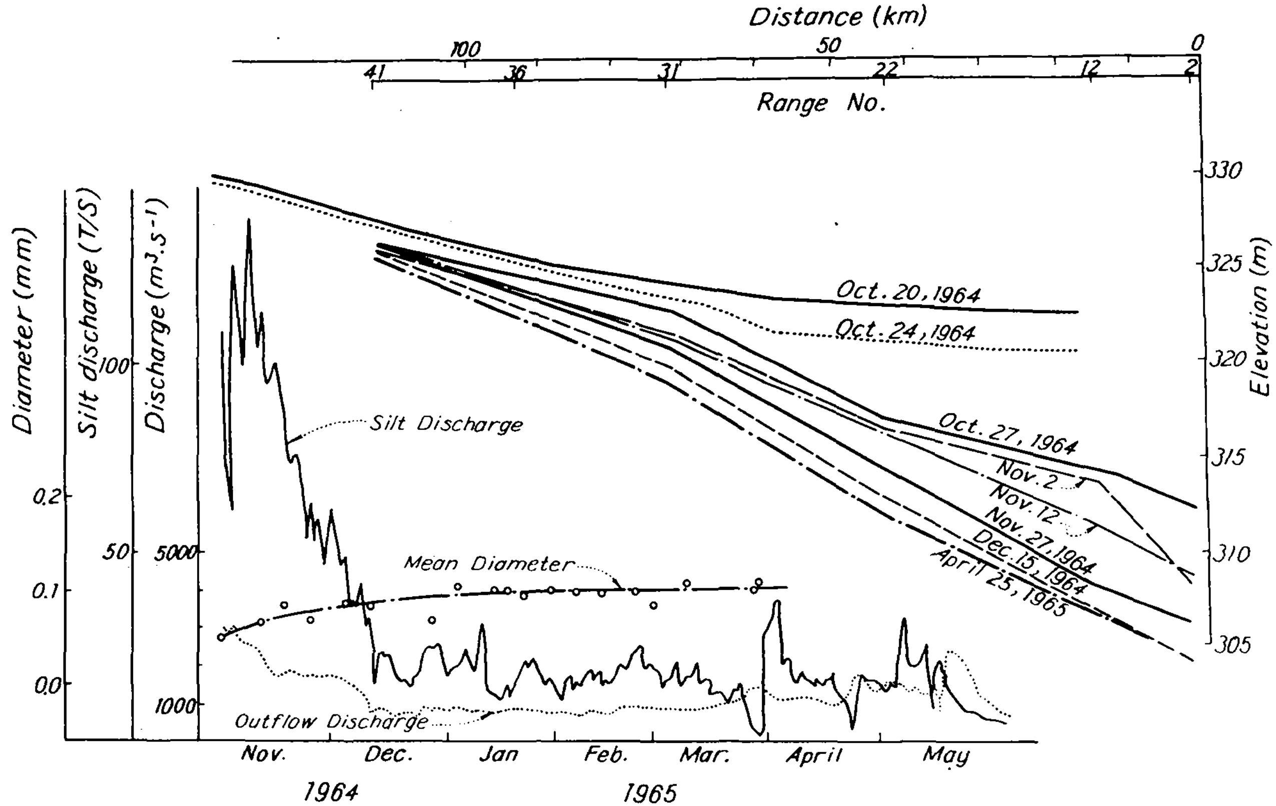


Figure 4. 21. Water level variation during retrogressive erosion in Sanmenxia Reservoir, 1964

During retrogressive erosion, the width of the main eroded channel at the water surface depends on the magnitude of the water discharge, as in stable channels for irrigation and as given by geomorphic relations between width and flow for rivers. The relationship between the width and the discharge is shown in Figure 4.22.

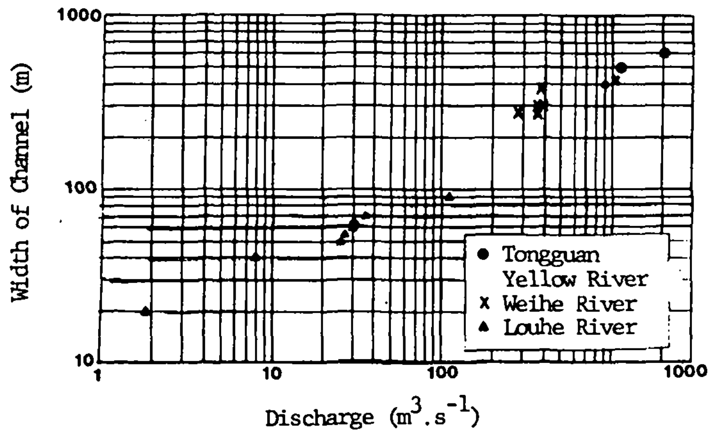


Figure 4.22. Relationship between the width of channel and the discharge

(4) Guanting Reservoir During the Detention Period

The Guanting Reservoir served as a detention reservoir in 1954. Figure 4.23 shows hydrographs of inflow and outflow discharge, inflow and outflow concentration, and mean diameter, as measured in the Guanting Reservoir. The transverse cross sections of the main channel formed by retrogressive erosion are shown on the right side of the figure. It is observed that when the water level fell, erosion developed with increases in the concentration and mean diameter. and a deep channel was formed after the erosion.

(5) Naodehai Detention Reservoir

Naodehai Reservoir is a gorge-type retention reservoir on Liuhe River in an arid region in the northeastern part of China. The dam was installed with two rows of gateless outlets, one of which was situated near the river bottom. By 1970, control gates were installed to preserve clearer water for irrigation in the nonflood season. Before 1970, the main purpose of the reservoir was for flood control. According to the design, an inflow flood of 3500 m^3s^{-1} could be regulated to 1640 m^3s^{-1}, and a flood of 5000 m^3s^{-1} at the downstream reach of the river could be reduced to 3500 m^3s^{-1} to prevent disaster in the lower reaches of the Liuhe River.

The heavily silt-laden Liuhe River has a yearly average concentration of 77 kg/m^3. During the flood season, the incoming heavy floods were detained. As the water level rose wihin the reservoir, siltation produced flood plain deposits due to the reduction of the flow velocity. When the water level was

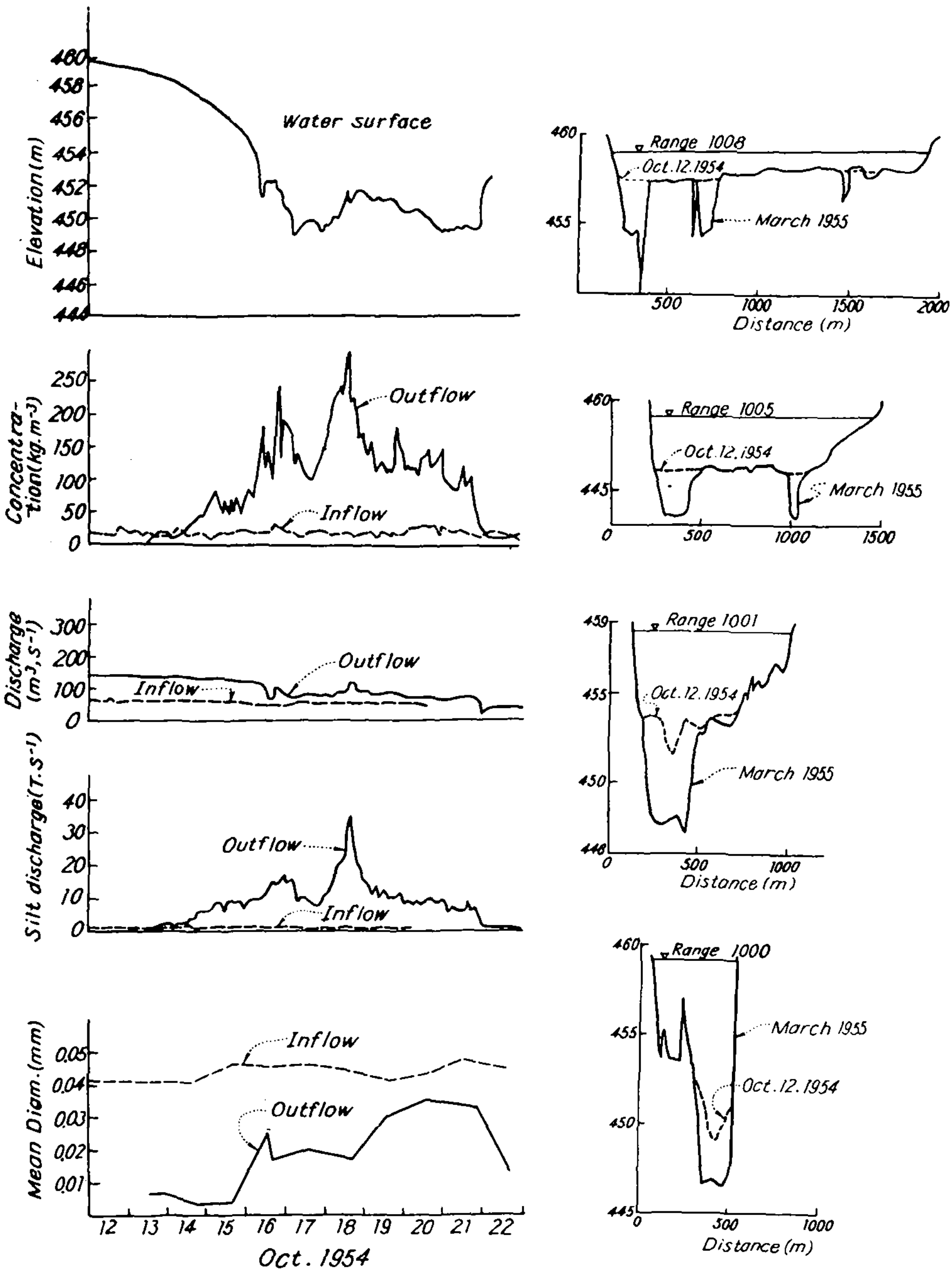


Figure 4.23. Drawdown flushing during detention period in Guanting Reservoir, 1954

lowered rapidly after the peak of the flood had passed, strong headward migrating retrogressive erosion occurred with outflow at a high concentration. The water and sediment discharges associated with three extreme flood peaks in July 1963 are indicated in Figure 4.24. Details of the considerable quantity of sediments detained are shown in Table 4.8. The flood phase was one of two extraordinarily heavy flood seasons which occurred in 1949 and 1963 after the completion of the dam.

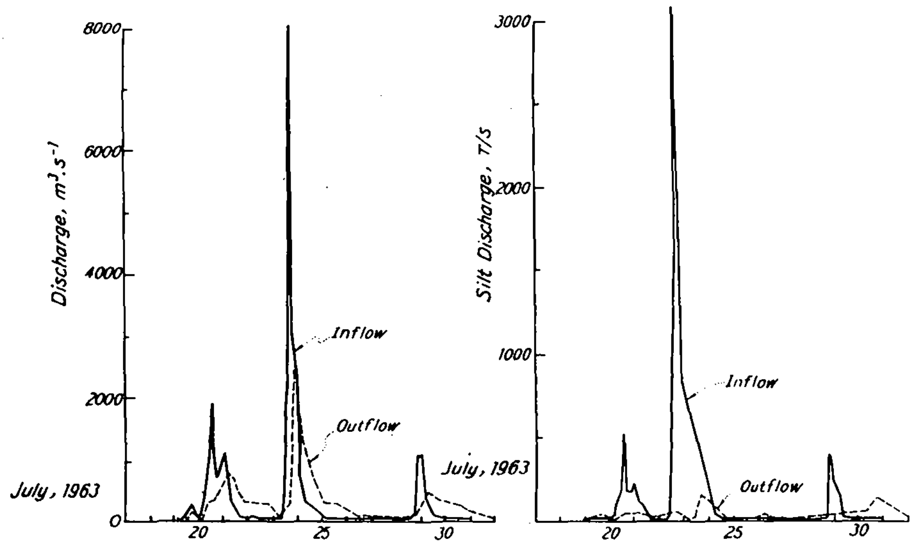


Figure 4.24. Inflow and outflow of discharge and silt discharge in July 1963, Naodehai Reservoir

It is interesting that the silt quantities released during each of the three floods were in the same order of magnitude of about 10 X 10^6 T.

The reservoir volume measured by the range method between 1942 and 1972 (Figure 4.25), showed periodic changes. It decreased in the years of heavy floods (1949 and 1963) and increased gradually after each of these two years. The volume of the reservoir was recovered primarily by retrogressive erosion, after 1949 and 1963, when the flow cut into the floodplain deposits and sediments were released from the reservoir.

The long-term useful storage capacity of a reservoir consists mainly of the volume of the main channel below the elevation of the floodplain, which may be maintained by regulating the flow and sediments in the reservoir (Fan and Jiang, 1980). The changes in the storage capacity of Naodehai Reservoir, shown in Figure 4.25, reflect a process of channel enlargement and deepening resulting from retrogressive erosion year after year. This may be further illustrated by Figure 4.26, which shows the vertical variation of the channel thalweg in selected years after 1949. The channel bottom was lowered by the process of successive erosion for 8 to 9 years, and approached the thalweg of 1940 before the completion of the dam.

Table 4.8. Quantity of deposits during flood peaks in 1963, Nadehai Reservoir

Date of flood	July 20-22	July 23-27	July 28-31
Max. water level (m)	84.57	88.62	83.02
Max. inflow discharge	1928	7980	1160
Max. outflow discharge ($m^3.s^{-1}$)	760	2470	440
Max. silt discharge of inflow	617	3280	388
Max. silt discharge of outflow (T/S)	67	168	147
Silt quantity in inflow	23.0	67.3	12.9
Silt quantity in outflow (10^6 T)	9.9	11.3	10.4
Quantity of sediment deposits (10^6 T)	13.1	56.3	2.5

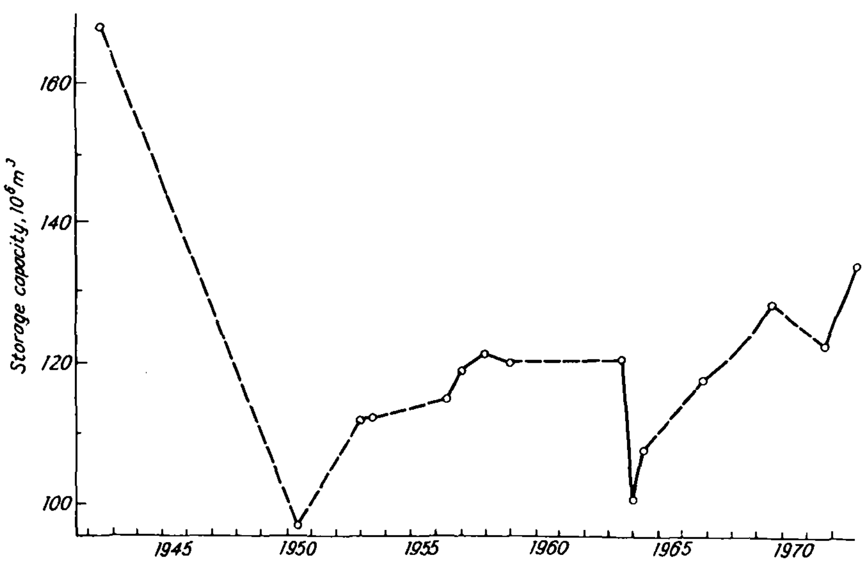


Figure 4.25. Time variation of storage capacity, Naodehai Reservoir

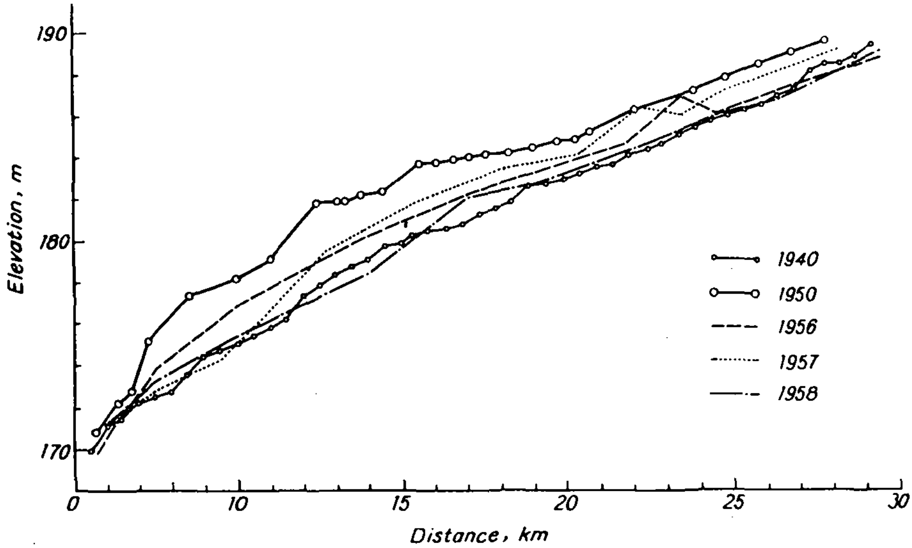


Figure 4.26. Thalweg in Naodehai Reservoir

Although successful channel floor erosion occurred, the flood plain elevation in the reservoir could not be lowered and indeed continued to show accretion as in Figure 4.27.

A model of retrogressive erosion and the methods of its computation have been developed (Fan and Jiang, 1980). This method may be used in computing the process of the silt quantity eroded from the reservoir under the conditions of a sudden drawdown of water level.

4.2.2 Drawdown Flushing

Drawing down the water level in a reservoir for the sake of reducing the amount of sedimentation, or in order to induce erosion of the deposited sediment to recover storage capacity, is a method often used in reservoirs, especially those of hydroelectric power stations. The efficiency of sediment flushing depends on the topographic position of the reservoir, the capacity of the outlet, the outlet elevation, the characteristics of the inflowing sediment, the mode of operation, the time duration of flushing, the flushing discharge, etc. Five examples will be given.

(1) Reservoir sedimentation in the Ouchi-Kurgan hydropower plant and reservoir (Zyrjanov, 1973)

The 17 km long Ouchi-Kurgan Reservoir in the USSR has (Figure 4.28a) a total storage capacity of 56.4 X $10^6 m^3$ and dead storage of 20 X $10^6 m^3$. The project is for the purposes of irrigation and power production. The dam has 8 bottom outlets, 20.8 m below the elevation of the power intake, which have a discharge capacity of 350 $m^3.s^{-1}$ with a maximum head of 35 m.

In order to maintain the useful storage capacity, a lowering of the pool level by 5 m during the flood season was considered in the design stage, because the storage capacity is comparatively small in comparison with the inflowing sediment discharge. The annual sediment discharge is between 12 X 10^6 and 14 X 10^6 tons.

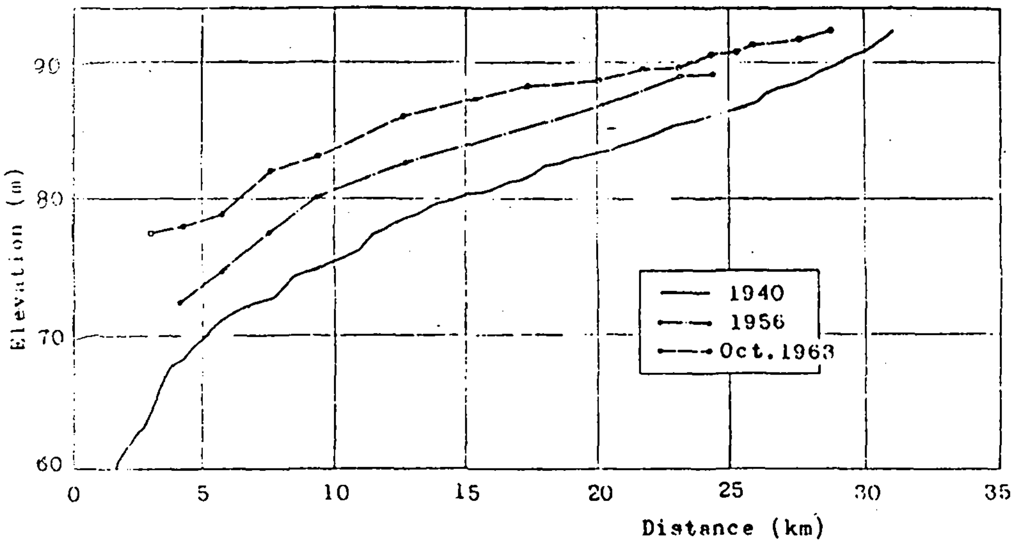


Figure 4.27. Profiles of flood plain, Naodehai Reservoir

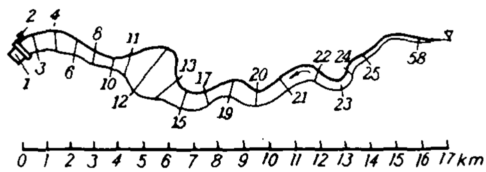


Figure 4.28a. Plan of Ouchi-Kurgan Reservoir (After Zyrjanov, 1973)

1. Dam
2. Gauge station
3-58. Range number

The reservoir began impounding in October 1961. Since 1963, the reservoir has been operated by lowering the pool level 4-5 m during the flood season (from May to August) every year, in order to reduce the siltation in the reservoir. The variation of water level, inflow suspended load, inflow discharge, and volume of deposits in the reservoir are shown in Figure 4.28b. No further loss of storage occurred after 1968, as indicated by line 4 in Figure 4.28b. The profiles of sediment deposits in the reservoir are shown in Figure 4.28c.

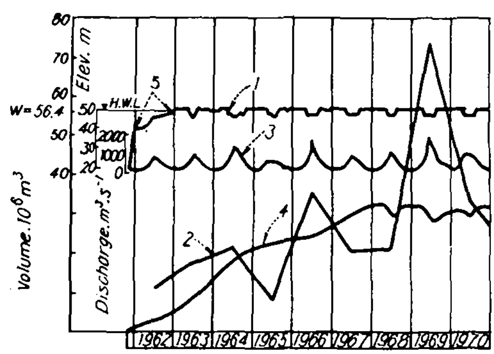
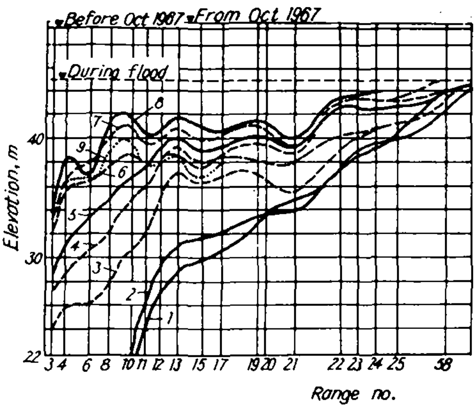


Figure 4.28b. Sediment volume deposited in Ouchi-Kurgan Reservoir (After Zyrjanov, 1973)

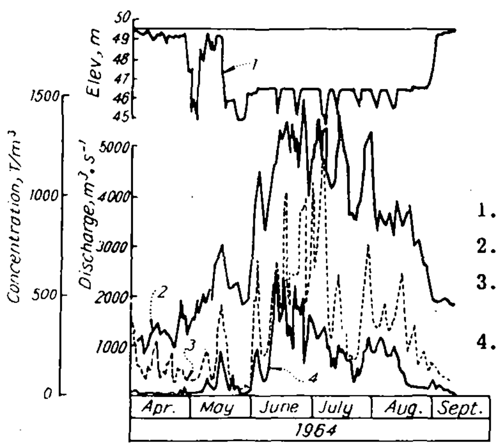
1. Water level
2. Annual volume of suspended load of inflow
3. River discharge
4. Volume of deposition in reservoir



1. 1957
2. June 1962
3. Sept. 1963
4. Oct. 1964
5. Oct. 1965
6. Sept. 1966
7. Oct. 1967
8. Oct. 1968
9. July 20, 1969

Figure 4.28c. Changes of reservoir deposition, Ouchi-Kurgan Reservoir

Figure 4.28d shows that the silt concentration of the outflow was less than that of the inflow, and that no erosion occurred in the reservoir. In other words, the flushing efficiency was not entirely satisfactory.



1. Water level
2. Discharge
3. Silt concentration of inflow
4. Silt concentration of outflow

Figure 4.28d. Hydrographs of water level, discharge and silt concentration, 1944, Ouchi-Kurgan Reservoir

The optimum discharge for sediment sluicing, to be regulated for a short period during the drawing down of pool water, was estimated to be more than 1,000 to 1,500 $m^3.s^{-1}$ for the Ouchi-Kurgan Reservoir.

(2) Zemo-Afchar Reservoir (Gvelessiani et al., 1968)

The Zemo-Afchar Reservoir-Hydroelectric Station located downstream from the confluence of the Kura and Aragvi Rivers, USSR, was completed in 1927. The reservoir, whose plan is given in Figure 4.29a, had a backwater length of 8 km in the reach of the Kura River and 1.8 km in the Aragvi River.

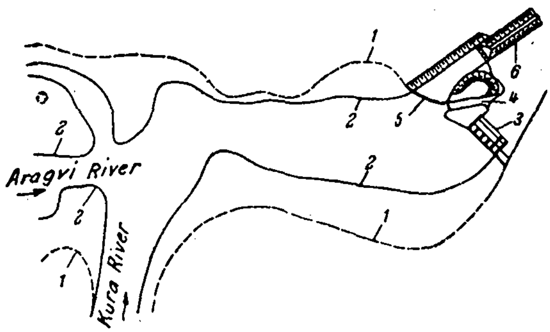


Figure 4.29a. Plan of the Zemo-Afchar Reservoir (After Gvelessiani, 1968)

1. Original river bank,
2. River bank in 1967
3. Dam
4. Diversion canal
5. Water intake
6. Canal

The two bottom outlets with a total width of 15 m are used for hydraulic flushing. The average annual discharge at the dam site is 210 $m^3.s^1$, and the suspended load approximates 4 X $10^6 m^3$, while the bed load provides more than 300,000 m^3.

After 40 years of use, probably with incorrect operation, 80 percent of the reservoir capacity had been lost. During the first two years, the storage capacity fell by 22 percent per year, and during the following 8 years a further 32 percent of the capacity was lost. In the period from 1937 to 1954, i.e., 18 years, further depletion of 3.5 percent of the initial storage capacity took place (Figure 4.29b). Prior to 1940, the water level was partially drawn down by about 2.3 m, but the flushing was not effective.

It was suggested that by full drawing down of the pool level, deposited sediments might be removed from the reservoir. In each flushing event there were two stages, partial drawing down followed by total drawing down.

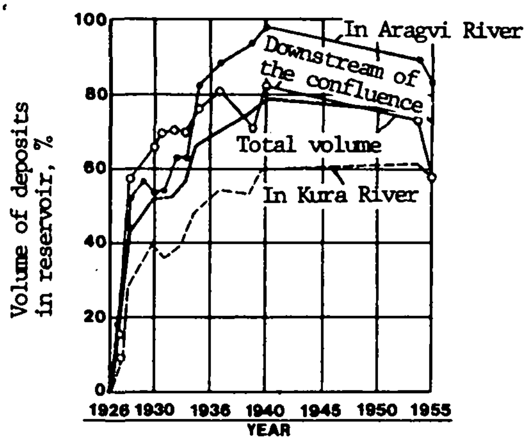


Figure 4.29b. Accumulative curves of siltation in Zemo-Afchar Reservoir in the period between 1926 and 1955

Examples of the variation of sediment concentrations and of the volumes removed during such hydraulic flushing on July 9, 1961, (Q = 130 m^3/s) and June 3, 1962, (Q = 285 m^3/s) are given in Figure 4.29c. The quantities of sediment evacuated are shown in Figure 4.29d.

In the first stage of the July 9 event, the time averaged rate of flushing was 10,500 m^3/hr, while on June 3 it was 4,000 m^3/hr; in the second stage of both events it was 54,000 m^3/hr, i.e., many times greater.

Some details of the flushing achieved in different years are presented in Figure 4.29d. On several occasions the most effective flushing did not occur immediately, but was delayed. High efficiency of flushing occurred immediately when the outlets were opened completely while the water level was being lowered.

The decision to change the operation to use flushing under conditions of complete drawdown of the water level is shown to be correct by the data in Figures 4.29c and 4.29d. It can be seen from Figure 4.29d that a river discharge of 400 to 500 m^3s^{-1} provides the optimum conditions for flushing the deposited sediments.

Detailed data from 38 periods of hydraulic flushing under conditions of full drawdown of the pool level in the reservoir are presented in Table 4.9. About one million m^3 of sediments (from 0.5 to 2 million m^3) were eroded on the average each year.

General conclusions may be drawn from the operation of the Zemo-Afchar Reservoir:

a. An optimum flushing discharge between 400 and 500 $m^3.s^{-1}$ produces the most effective evacuation of sediments. When the flushing

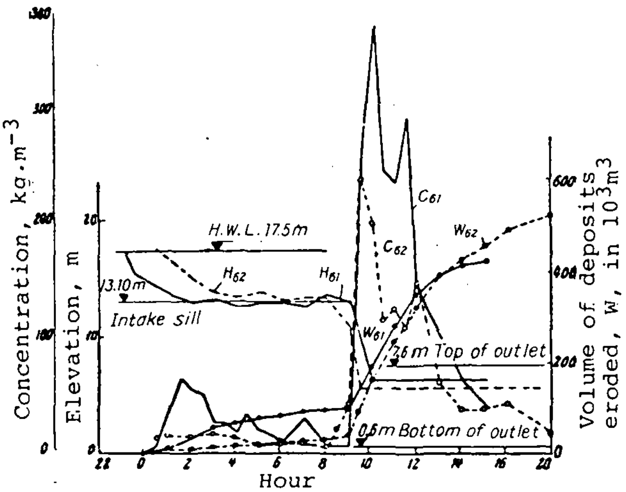


Figure 4.29c. Variation of water level, silt concentration and quality of deposits eroded

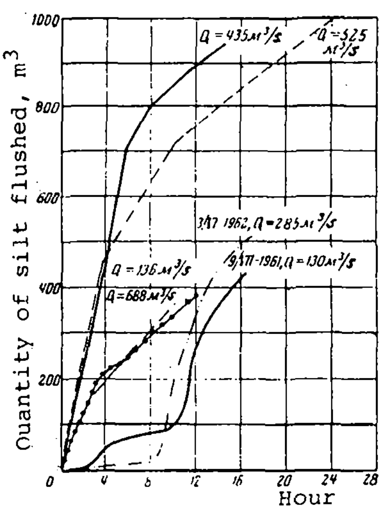


Figure 4.29d. Changes of the silt quantity eroded in Zemo-Afchar power station

Table 4.9. Data of sediment flushing by drawing down of water level in Zemo-Afchar Reservoir

Date of flushing	River discharge, $m^3.s^{-1}$	Flushing discharge, $m^3.s^{-1}$	Volume of eroded deposits, 10^6m^3	Volume of water used for flushing, 10^6m^3	Duration of flushing hr	Mean sediment concentration, kg/m^3	Volume of sediments eroded in one year, 10^6m^3
Nov. 7-8, 1939	325	325	0.67	35.0	30.0	-	0.67
Nov. 7-8, 1940	205	205	0.80	-	-	-	0.80
May 1-2, 1941	930	688	0.74	50.1	20.5	22.6	0.74
Apr. 8-9, 1945	500	480	0.81	51.8	30.0	24.2	0.81
May 13-14, 1947	217	217	0.49	26.0	34.0	29.8	0.49
July 4-5, 1948	127	127	0.74	17.2	37.4	66.7	0.74
May 14-16, 1949	742	592	0.69	78.7	37.0	13.1	-
Nov. 13, 1949	89.0	89.0	0.25	3.60	11.2	109	0.94
Apr. 16-17, 1950	433	431	0.68	38.8	25.0	27.0	-
Nov. 8, 1950	110	110	0.40	3.37	8.50	182	1.03
May 13-14, 1951	255	251	0.56	24.9	26.5	34.9	-
Nov. 11, 1951	202	193	0.46	7.83	11.0	91.3	1.02
Apr. 23-24, 1952	525	525	1.01	46.5	24.5	33.7	-
July 2, 1952	435	435	0.94	22.7	14.5	64.2	1.95
Apr. 26-27, 1953	443	443	1.13	41.4	26.0	42.1	-
Nov. 3, 1953	90.0	90.0	0.33	4.02	12.0	126	1.46
Apr. 25-27, 1954	900	728	0.96	171	65.5	8.61	-
Nov. 2, 1954	75.0	72.0	0.29	3.00	11.6	152	1.25
May 22, 1955	381	381	0.68	16.0	11.0	68.6	0.68
Apr. 22, 1956	800	633	0.54	31.9	14.0	26.2	-
Nov. 7, 1956	136	136	0.39	6.01	12.25	100	0.93
Apr. 28, 1957	436	436	0.52	20.4	13.0	39.9	0.52
Apr. 27, 1958	412	457	0.50	18.3	11.2	42.2	-
July 27, 1958	112	112	0.38	4.24	10.5	140	0.88
Apr. 12, 1959	750	547	0.47	31.4	16.0	23.3	-
Nov. 8, 1959	130	134	0.48	5.0	10.3	150	0.95
Apr. 24, 1960	1070	648	0.36	29.7	13.0	19.0	-
Oct. 2, 1960	108	126	0.49	5.58	12.3	13.7	0.85
Feb. 12, 1961	80.0	83.9	0.06	2.12	7.0	46.5	-
Apr. 23, 1961	470	395	0.30	18.1	12.7	25.4	-
July 9, 1961	130	163	0.42	9.42	16.0	69.2	-
Oct. 1, 1961	97.0	97.0	0.16	4.10	11.8	61.8	0.94
July 3, 1962	285	285	0.52	13.5	10.0	59.7	-
Nov. 6, 1962	98.0	116	0.24	4.6	11.0	79.1	0.76
Apr. 21, 1963	460	439	0.64	21.6	13.7	29.6	0.64
May 17-18, 1964	800	437	0.75	62.1	39.5	16.2	0.75
May 23, 1965	574	553	0.46	21.2	13.4	21.9	0.46
May 22, 1966	506	402	0.64	36.0	24.8	24.0	0.64

discharge is greater than the optimum value its effectiveness decreases due to the existance of backwater effects. If the flushing discharge is less than the optimum, erosion is decreased because the stream power of the flushing flow is below its peak value.

b. During the process of sediment flushing, the most active erosion occurs in a period of 8 to 10 hours after the practical erosion begins.

c. A decrease in the effectiveness of flushing may be restored by a partial or full impounding of water for a short duration followed by complete drawing down of the water level.

(3) Flushing operation in Khashm El Girba Reservoir (El Hag, T., 1980, El Fatih Saad, A., 1980)

The Khashm El Girba dam was completed in 1964, on the Atbara River in Sudan. Its original storage capacity was 950 X $10^6 m^3$ and the water was used for irrigation, hydroelectric power and water supply. The capacity was seriously depleted by silting of the reservoir as a result of the average annual sediment inflow of about 84 X 10^6 tons.

Some details of the drawdown flushing operations carried out in July 1971 and 1973 are listed in Table 4.10.

Table 4.10. Flushing operation in Khashm El Girba Reservoir

Period	Inflow volume of water ($10^6 m^3$)	Inflow silt discharge ($10^6 T$)	Net sediment releases ($10^6 m^3$)	Sediment of outflow / Vol. of water of outflow
July 11-14, 1971	612	3.5	17.5	0.0286
July 29-Aug. 2, 1973	545	3.3	12.5	0.023

According to El Hag (1980) and El Fatih Saad (1980) the sediment outflow in each July, including the flushing operation periods, was 85 X 10^6 tons, which is greater than the estimated average annual sediment inflow. Unfortunately, no further details are available.

(4) Shuicaozi Reservoir

The Shuicaozi Reservoir and Hydroelectric Power Station is located in southwest China. The storage capacity of the reservoir is 9.58 X $10^6 m^3$; the dam height is 36.9 m; and the length of the reservoir is 6 km.

The yearly mean discharge at the dam site is 16.3 $m^3.s^{-1}$ and the discharge for power production is 29 $m^3.s^{-1}$. Inflow is regulated by an upstream reservoir. The suspended load is estimated to be 600 X 10^3 tons p.a. and the bed load amounts to 30,000 tons p.a.

In order to use the water coming from the drainage area between the upstream dam and the Shuicaozi Reservoir and from the upstream dam, a useful storage of 3,600,000 m^3 was required. Between June 1958 and January 1981, sediment deposited in the reservoir totaled 8,180,000 m^3, i.e., 85 percent of its total storage capacity. The remaining 1,400,000 m^3 of volume was not enough for flow regulation.

Since 1965, experiments have been conducted on drawing down the water level in the Shuicaozi Reservoir to erode deposits from the reservoir. The outflowing water passes down the spillway, the top elevation of which is at 2089 m. The normal high water level is 2100 m and the intake to power station is at 2088 m. Drawing down the water level is, therefore, limited by the high elevation of the spillway. The flushing is often carried out for two or three days during the Spring Festival when the power station is not working.

A water discharge of about 50 $m^3.s^{-1}$ is especially released from the upstream reservoir and used for the drawdown flushing. The elevation of the spillway limits the lowest water level behind the dam to about 2090 m. The effective time interval for each period of flushing is less than one day. The maximum sediment concentration recorded is not more than 200 kg/m^3. Details of discharges during flushing by drawdown of the pool level or emptying of the reservoir in 1965, 1966, 1974, 1978, 1980 and 1981 are given in Table 4.11. It can be seen that in one period of erosion, 1 to 2 days, 200,000 m^3 of sediments may be eroded out each year. This corresponds to one third of the annual incoming silt discharge. Behind the dam two meters of erosion occurred, but no erosion took place 4 km upstream of the dam.

The quantity which could be evacuated is limited partly because the fine sediment deposits have become consolidated, partly because deposition of the bed load occurs in the upper part of the reservoir, and partly due to the high elevation of the spillway through which the flushing discharge must pass.

Longitudinal profiles of the thalweg before and after flushing are given in Figure 4.30. Discharge and sediment concentration variations during drawdown flushing are shown in Figure 4.31.

(5) Sediment withdrawal in Guernsey Reservoir (Jarecki, 1965)

The Guernsey Dam on the North Platte River, USA, was completed in 1927. The reservoir is used for the storage of irrigation water, reregulating upstream storage releases, and for power generation. The storage capacity at a crest elevation of 1347 m is 91 X $10^6 m^3$. The length of the reservoir is 23.5 km. The drainage area above the Guernsey Dam is about 42,000 km^2, the sediment contributing area has varied from 14,200 km^2 in 1927 (below Pathfinder Dam), to 14,000 km^2 in 1938 (below Alcove Dam) and to about 1,800 km^2 in 1957 (below Glendo Dam).

Before the dam construction, the irrigation system in the North Platte area received a heavy intake of sediments in the diverted canal waters for many years. Canal cleaning problems were severe, but canal seepage losses appeared to be low. With the construction of Guernsey Dam and Reservoir in 1927, clear water releases tended to increase the canal seepage losses and canal bank erosion.

Until 1957, the reservoir was subject to a very high sedimentation rate, losing 39.3 percent of the original capacity in thirty years of operation.

The reservoir was drawn down purposely to produce increased sediment concentration in the releases.

Table 4.11. Data observed during drawdown flushing in the Shuicaozi Reservoir

Duration	Day of emptying of reservoir	Min. water level (m)	Discharge of flushing (m^3/s)	Max. con-concentration of outflow (kg/m^3)	Quantity of water used ($10^3 m^3$)	Quantity of sediment flushed out ($10^3 m^3$)	
						Silt discharge method	Range method
June 15, 14:00- June 16, 18:00, 1965		2090	44 - 175	38.3	11,500		133
Aug. 31, 10:00-14:34, 1966		2090.8	200 - 230	140	3,240		138
Spring Festival of 1974			90 - 97.5	87.9	10,000		275
Feb. 5, 16:10- Feb. 8, 18:22, 1978	3	2092.7	40 (max.)	129.8	4,250	109	175
Feb. 14, 8:20- Feb. 19, 3:00, 1980	4	2089.9	21.4 - 37.9	165	15,740	256	316
Feb. 4, 21:49- Feb. 7, 9:00, 1981	3	2089.9	58.6	79	8,060	120	181

* A specific weight of 1.2 for the deposited sediment has been assumed in the computation.

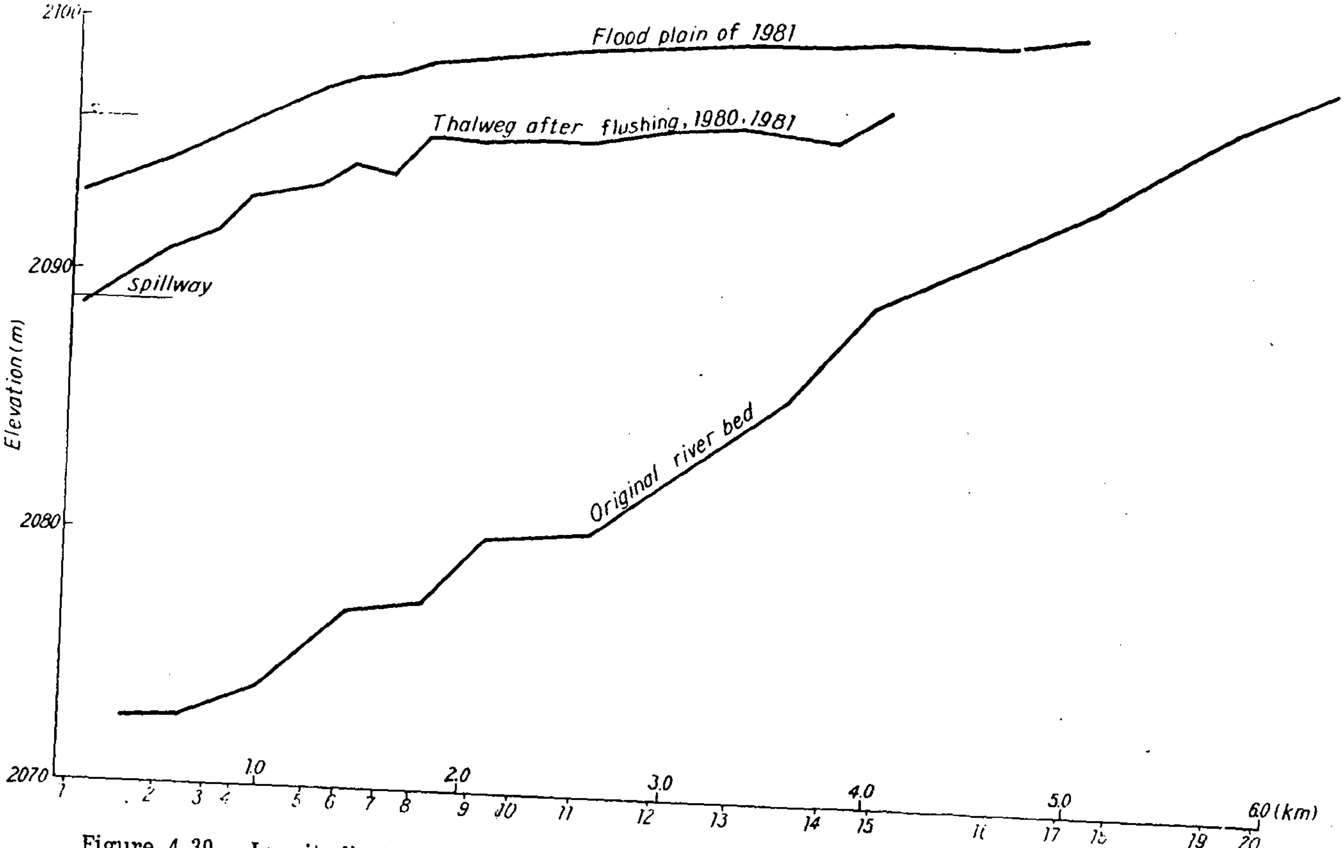


Figure 4.30. Longitudinal profiles of the thalweg before and after flushing, Shuicaozi Reservoir

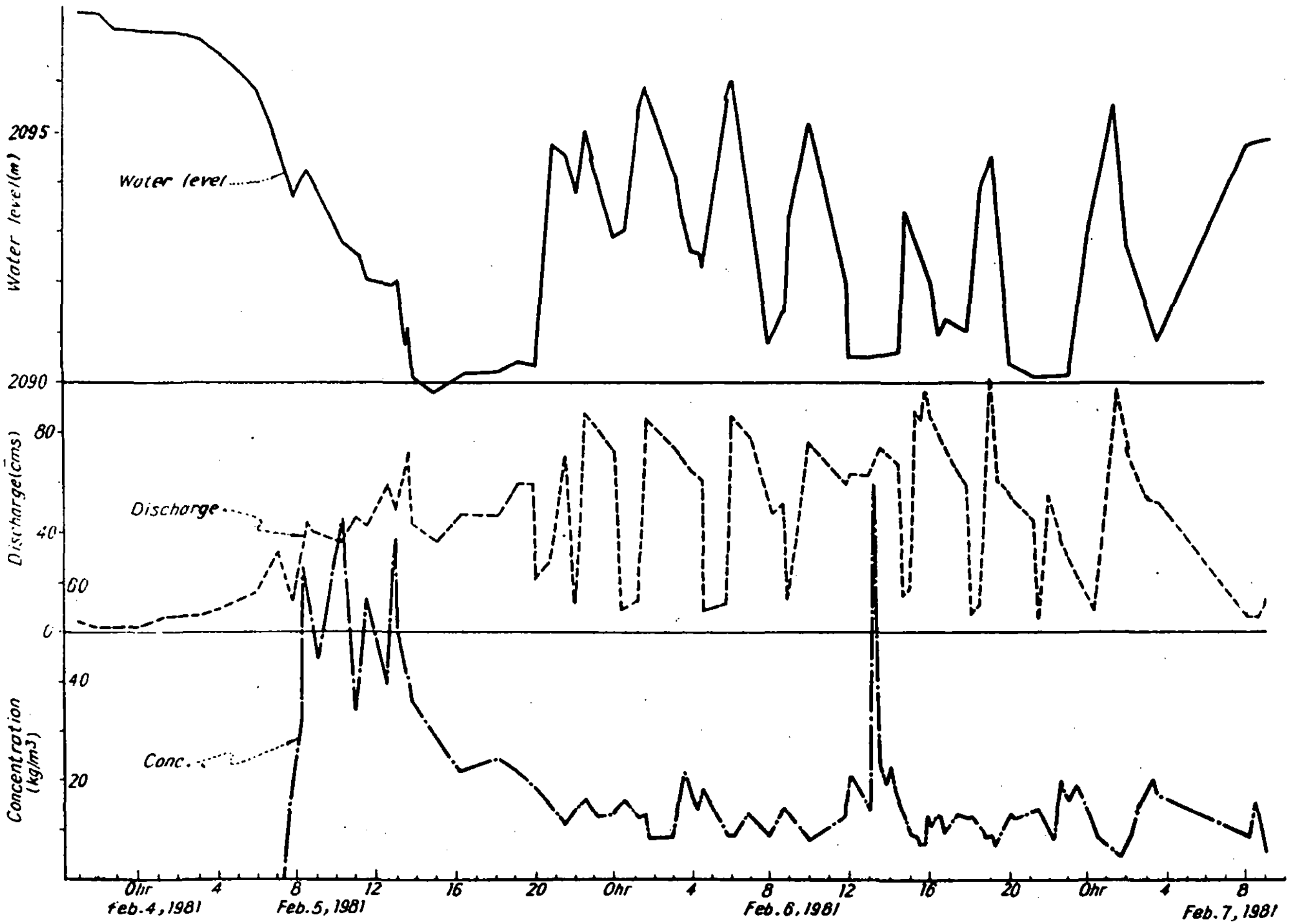


Figure 4.31. Discharge and sediment concentration variation during drawdown flushing, Shuicaozi Reservoir

During the drawdown the reservoir inflow ranged between approximately 56.6 and 198 $m^3.s^{-1}$ with controlled releases from the upstream Glendo Reservoir. Experiments showed that sediment withdrawal was greater when water levels were low, i.e., during reduced reservoir storage conditions. .

The effect of storage on the sediment concentration of reservoir releases is shown in Figures 4.32 and 4.33. The longitudinal profiles of sediment deposits are shown in Figure 4.34.

From the measurements of the inflow and outflow sediment concentration, the outflow one was always smaller than the inflow, but erosion occurred at the bottom of the reservoir. Limited scour occurred during drawdown of water levels in the reservoir during 1960-1962 as indicated in Figure 4.34.

The ratio of the sediment release to the quantity of water releases was very low:

(1)	July 10-19, 1960	=	0.000174
(2)	Aug. 8-18, 1960	=	0.000122
(3)	July 20-Aug. 3, 1961	=	0.000163
(4)	July 24-Aug. 12, 1962	=	0.000249
	The average rate	=	0.00018

As far as the water quantity used for drawdown flushing is concerned, it seems uneconomical in the case of the Guernsey Reservoir.

Several reservoirs, mainly for power generation in the USSR, are operated in the manner of drawing down the water level to wash out the sediment deposits and to recover a part of the storage capacity (Qian, 1982). The water-detritus ratio ranges from 30 to 50, with a maximum value of 100. The flushing periods of one or two day duration observed in the Zemo-Afchar power station suggest flushing water-detritus ratios of 8 to 83 according to records between 1939 and 1966. The flushing water-detritus ratio becomes larger for flushing periods of three days (1949, 1954).

The efficiencies of flushing as represented by water-detritus ratios in the examples considered in this chapter and others, are summarized in Table 4.12.

It can be seen that more water has to be used for overflow spillways than for bottom outlet systems. It is apparent that as far as the water requirement for flushing is concerned, the flushing in Guernsey Reservoir is not economical; although the small concentration of outflow may decrease the canal seepage loss and canal bank erosion.

In the Shicaozi power station, the flushing is also not economical. The water used for sediment release was derived from an upstream power station, so that energy production is lost in the flushing process. If bottom outlets were available, it is certain that more sediment would be sluiced out and more storage capacity recovered.

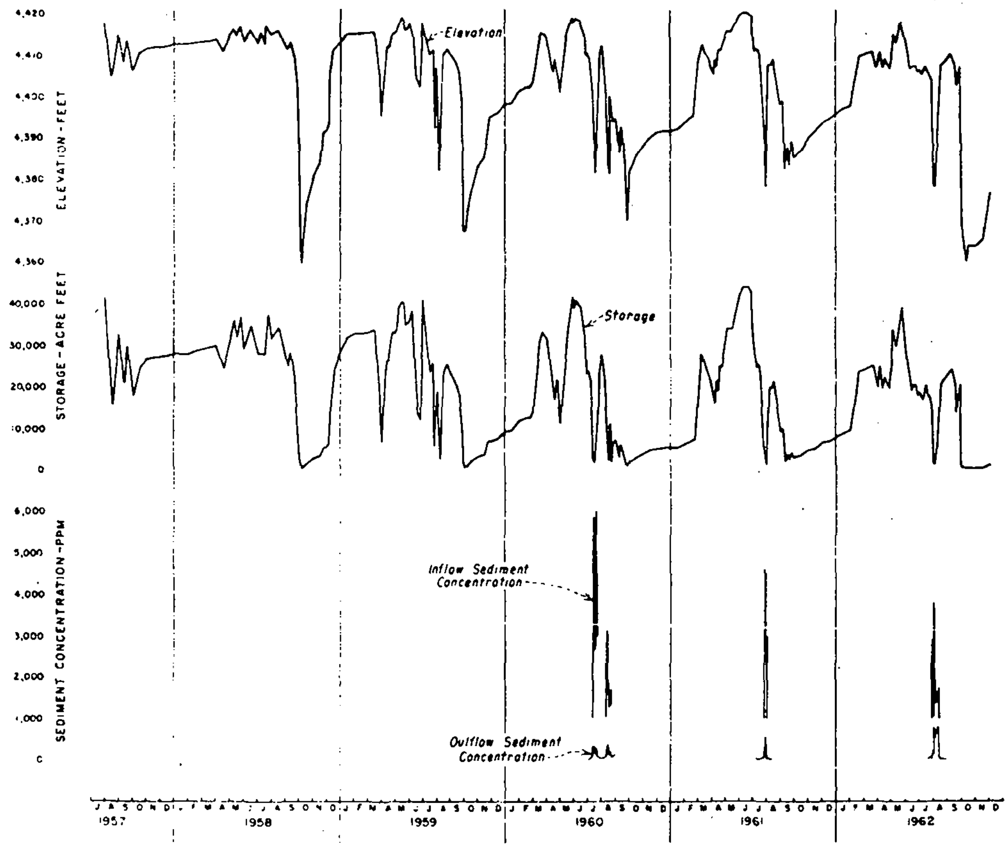


Figure 4.32. Elevation-storage-sediment concentration, 1957 to 1962, Guernsey Reservoir (After Jarecki, 1965)

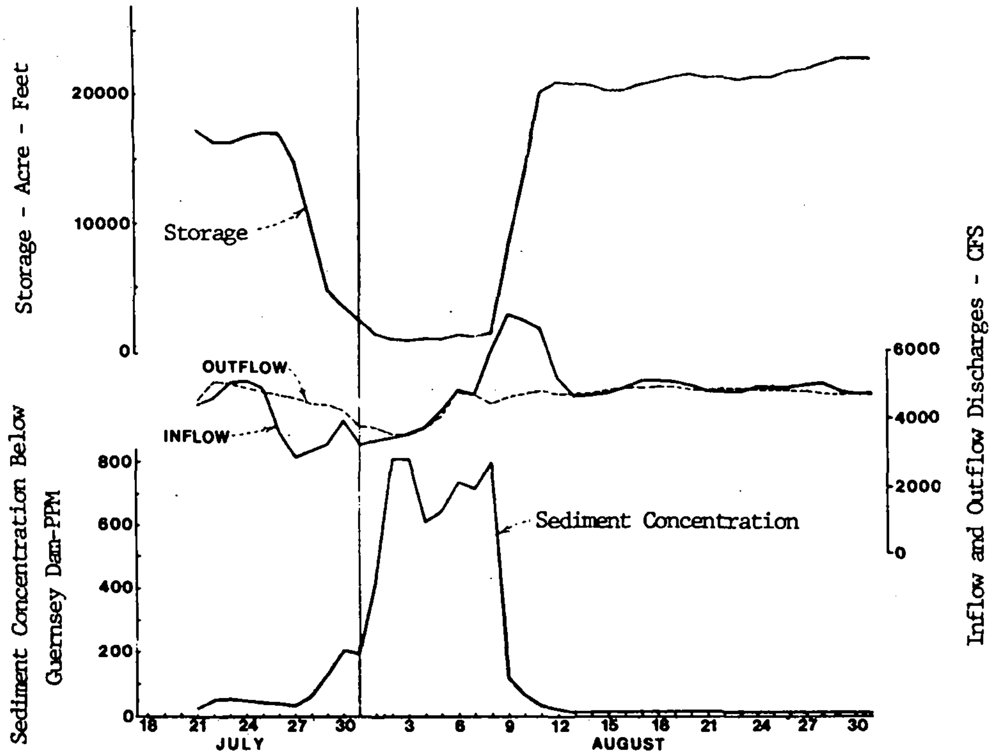


Figure 4.33. Guernsey Reservoir Drawdowns, 1962
(After Jarecki, 1965)

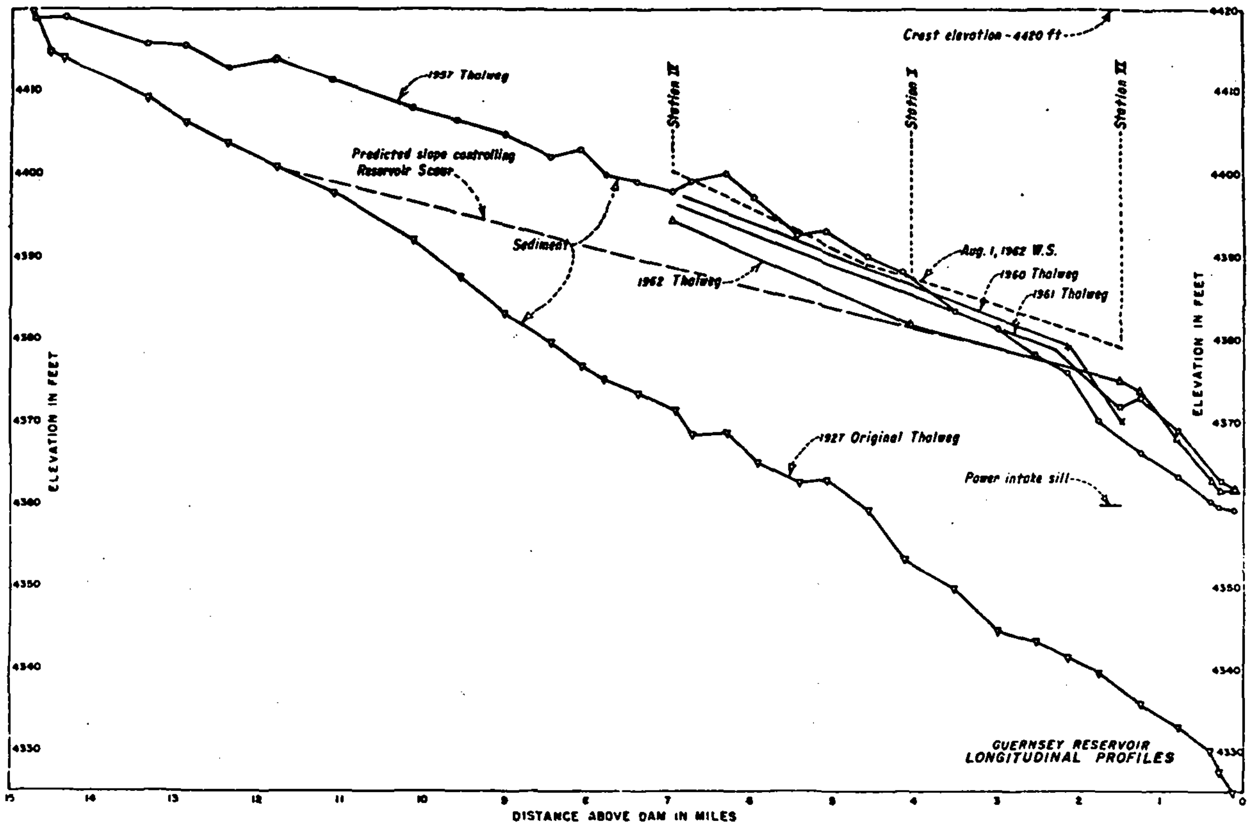


Figure 4-34. Longitudinal profiles of sediment deposits (After Jarecki, 1965)

Table 4.12. The efficiency of the operation of drawdown sediment releases

Reservoir	Mode of flushing	Ratio between water and solids	References
Santo Domingo	Reservoir emptied, bottom outlets	7-11	Krumdieck et al., 1979
Grimesl, Switzerland	Bottom outlet	16-22	Dawans et al., 1982
Kashim El Girba, Sudan		35-43	Table 4.10
Zemo-Afchar USSR	Bottom outlet	8-83 (for flushing period of 1-2 days)	Table 4.9
Shuicaozi, China	Overflow spillway	23-86	Table 4.11
Guernsey, USA	Overflow spillway	400-820	Jarecki, 1965

4.2.3 Density Current Flushing

The venting of density currents has long been considered an effective means of relieving the rate of reservoir silting, especially in impounding reservoirs.

Qualitative laboratory flume tests on the possibility of venting density currents were performed in the twenties, making use of turbid water to form the density current (Smrček, 1929) and of saline water instead of cold water (Schoklitsch, 1929). These experiments were in accordance with occurrences observed at lakes and reservoirs (Forel, 1885). The front of a turbid density current in a laboratory flume is shown in Figure 4.35.

Field observations have been made in many reservoirs following the recognition of the phenomenon of density currents. Some travelled more than 100 km before being vented out through the diversion outlet in Lake Mead, USA, revealing the potentiality for venting density currents in restricting siltation in reservoirs, e.g., the Sautet Reservoir in France, the Metka and the Groshnitza reservoirs in Yugoslavia, and the Nulek reservoir in the USSR (Mihailova et al., 1975; Pyrkin et al., 1978).

In the next section some of the measurement data showing the existence of density currents in reservoirs will be described. The amount of sediment sluiced from the reservoir, as compared with the inflowing silt discharge, may be estimated according to the method developed based on the field measurement. In addition, the operational procedure for venting a density current from the reservoir will be discussed.

4.2.3.1 Features of density currents in Elephant Butte Reservoir. The recognition that density currents existed in the Elephant Butte Reservoir (Fiock, 1934) was based on several observations:

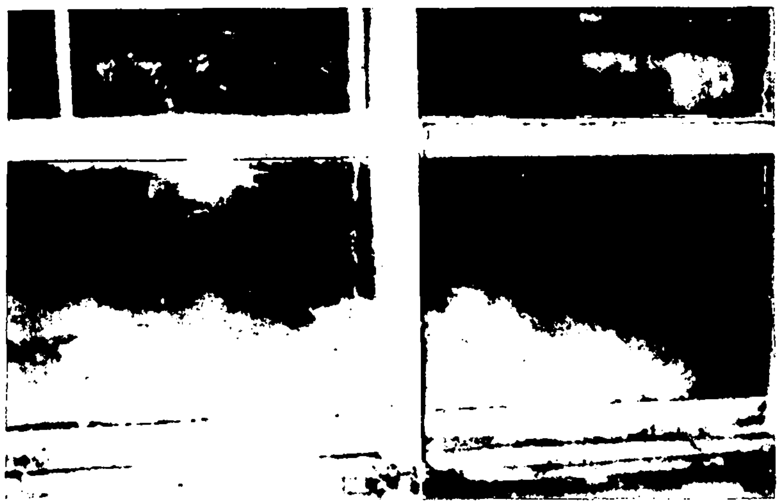


Figure 4.35. The front of a turbid density current in a laboratory flume 50 m long, 0.5 m wide and 2 m high. The mean thickness of the density current is about 0.23 m and the total water depth is 1 m. Photo Institute of Water Conservancy and Hydroelectric Power Research, Beijing, China, 1959.

(1) The sinking of the sediment-laden water below clear water occurred in an area of 21-23 km above the dam site. A sharp line of demarcation separated the muddy water from upstream and the clear water of the lower basin.

(2) Two to five days after the entrance of heavily sediment-laden flood waters (concentration of 4 to 10 percent by weight), into the upper end of the reservoir basin, turbid water with a concentration of 2 to 6 percent was discharged from the outlet gate in the dam. The turbid discharges usually continued for only a few days. The travel time of the density current from the head of the reservoir to the dam site was approximately 2 to 5 days.

(3) When the silt-laden water was being discharged from the outlet gates, the temperature of the outflow immediately rose about 6°F (3.3°C) and the soluble salt content increased materially. With the clarification of the outflow water which usually occurred quite rapidly when the temperature of the outflow returned to the normal temperature of the reservoir outflow for that particular season of the year, and the soluble salt content also fell back to that of ordinary reservoir water.

(4) More than 10 m of sediment had accumulated at the dam. From the records of the inflow discharge (mainly from the tributaries, Rio Puero and Rio Salodo) and of the outflow discharge, the similarities between them during specific events may be explained by the presence of density current movements travelling through the reservoir and venting out from it. They are set out in Table 4.13 (after Lane, 1954).

According to the measurements taken by Resch (Lane, 1954) at a cross section 300 m above the dam, the silt content, salt content and temperature vary in a vertical direction (see Table 4.14), verifying that the bottom density current is coming from the inflow discharge during the flood.

4.2.3.2 Lake Mead. Lake Mead is the reservoir formed by the construction of Hoover Dam on the Colorado River, USA. In 1935, it had a total capacity of 38.4 X $10^9 m^3$ (to the top of the spillway gates in raised position), and a usable capacity of 34.5 X $10^9 m^3$. The annual inflow averaged about 16 X $10^9 m^3$. The annual loads of suspended sediment at Grand Canyon averaged about 155,000,000 tons. Figure 4.36 shows the point of plunge of a density current in Lake Mead.

Figure 4.36. Plunging point of a density current. A motorboat approaching the point where muddy water from the Colorado River (lower right) disappears below the blue surface of Lake Mead. (Bureau of Reclamation, U.S.A.)

Table 4.13. Density currents vented out from Elephant Butte Reservoir

Inflow of silt discharge			Outflow of density current		
Date	Sediment wt per m^3 (kg)		Date	Sediment wt per m^3 (kg)	
	Average	Max.		Average	Max.
1917 several days	no records		1917 several days	no records	
1919 July 5-22	72	119	1919 July 7-10	41	41
July 31-Aug. 9	68	163	July 18-28	41	41
1921 July 23-29	97	124	1921 several days	no records in details	
Aug. 1-4	58	77			
1923 Sept.	no records		1923 Sept. 22-26	11	20
1927 Sept. 9	113	113	1927 Sept. 19-21	64	67
Sept. 15	40	40			
1929 Aug. 8-16	64	119	1929 Aug. 13-31	26	52
Aug. 25-Sept. 5	47	74	Sept. 26 - Oct. 7	47	67
1931 Sept. 19-25	98	115	1931 Sept. 22-25	46	58
1933 June 15-28	47	101	1933 June 23-30	35	54
1935 July 28	1	1	1935 Aug. 9-11	23	41
Aug. 5-6	68	78	Aug. 25-26	17	20
Aug. 18-22	80	139			
1941 May	no records		1941 May 9 several hrs	no records	

Table 4.14. Profiles of temperature, salt content and sediment concentration measured at C.S. 300 m above the dam

Elev. in Lake (ft)	Water depth (ft)	Water depth (m)	Temp. (°C)	Salt content (p.p.m.)	Silt content (p.p.m.)
4.335.8	Water surface		26.7	400	0
34.8	1	0.3	26.1		
30.8	5	1.5	25.6		
25.8	10	3.1	25.6		
15.8	20	6.1	25.0		
05.8	30	9.2	24.4		
4.295.8	40	12.2	23.3		
85.8	50	15.3	21.7	540	60
75.8	60	18.3	20.6		
70.8	65	19.8	18.9		
65.8	70	21.4	18.3		
60.8	75	22.9	17.8	600	100
55.8	80	24.9	17.8	600	0
54.8	81	24.7	17.8	600	0
54.3	81.5	24.9	18.3		
53.8	82	25	20.0	700	1,100
52.8	83	25.3	20.6	900	11,000
50.8	85	25.9	21.7	900	30,800
45.8	90	27.5	22.2	1,100	41,800
40.8	95	26.0	22.2	1,000	37,000
39.8	96	29.3	21.7	1,000	45,300
4.239.3	96.5 (bottom)	29.4			

Some 21 periods of density currents were observed in 1935 and 1936 (Grover and Howard, 1938) and others were reported in subsequent years by National Research Council, and Bureau of Reclamation, as shown in Figure 4.37 (Smith et al., 1960). An indication of the existence of a density current reaching the dam is the increase in the amount of sediment at the dam, with an increased elevation of the sediment surface.

From the hydrograph of the outflow and inflow of silt discharge, the ratio of outflow to inflow of suspended load was calculated to range from 0.18 to 0.39. The following characteristics of the density currents in Lake Mead may be noted:

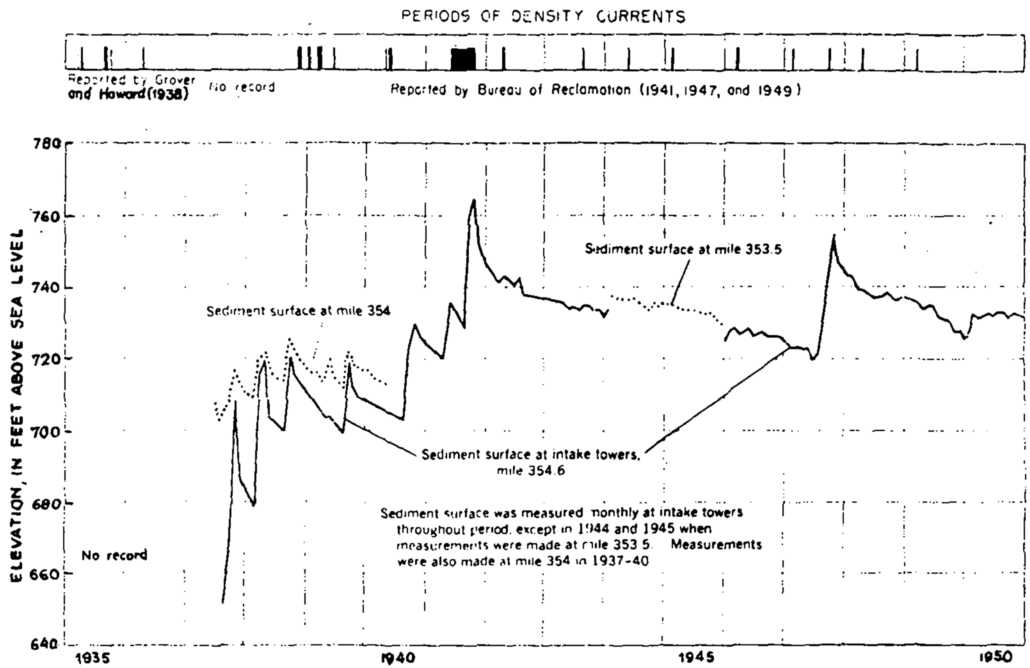


Figure 4.37. Periods of reported density currents and elevation of sediment surface at Hoover Dam, 1935 to 1950 (After Smith et al., 1960)

(1) The inflow concentration is an important factor in maintaining the travel of the density current along the bottom of the reservoir and to its venting out. An inflow concentration of less than 1.0 percent by weight seems to be insufficient to maintain enough motion for density currents to reach the dam in Lake Mead. See the flood peaks April, September, October, 1935 and April, 1936 (Figures 4.38 and 4.39).

(2) The concentration of inflowing sediments with diameters smaller than 0.02 mm at the Grand Canyon Station is plotted in Figure 4.38. This shows the

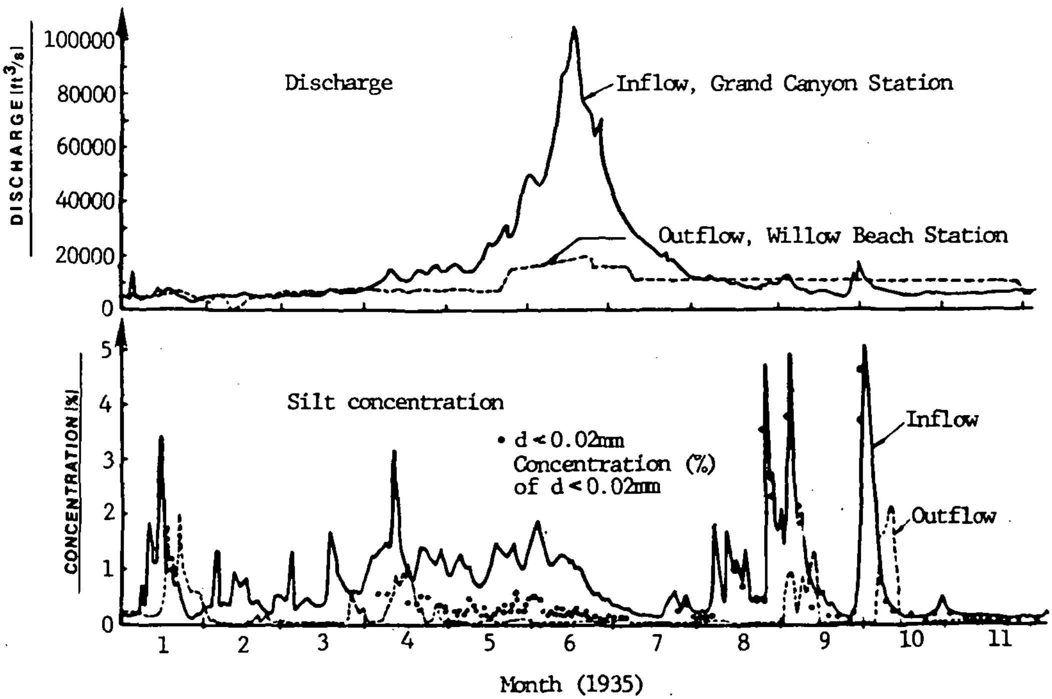


Figure 4.38. Hydrographs of discharge and silt concentration, 1935, Lake Mead

effect of the finer particles in maintaining the density currents. Note that apparently no silt was discharged past the Willow Beach Station from May to August, owing to the small quantity of fine particles contained in the inflow. However, in September and October the high proportion of fires ensures the presence of density currents.

4.2.3.3 Iril Emda Reservoir (Duquennois, 1959, Raud, 1958, a,b). The Iril Emda dam was built on the Oued Agrioun in Algeria for power production. It is about 61 m high and stores about 150 X $10^6 m^3$. The dam was put into operation in September 1953.

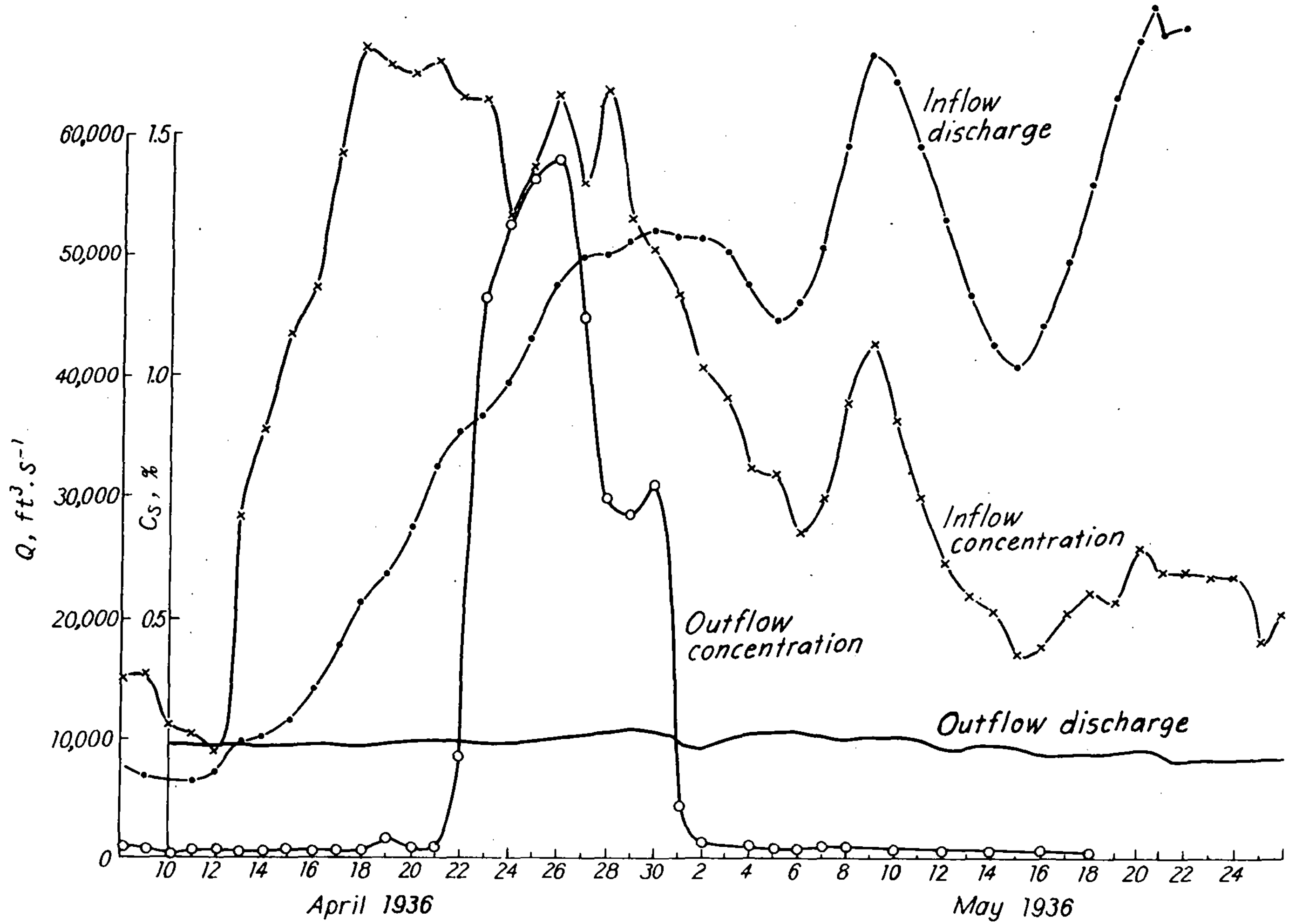


Figure 4.39. Hydrographs of discharge and silt concentration, April and May 1936

Eight small valves, 400 mm in diameter, bypassing the main scour sluices, are regulated by altering the number opened in order to avoid the frequent operation of the scour sluices.

Observations were made of the weight of sediments removed in every hydrologic year (September 1 to August 31). According to Duquennois (1959), during the period of 1953 to 1958, between 45 percent and 60 percent of the inflow of sediment was vented out. Only 25 percent of the inflowing sediment was evacuated in the first year of operation, partly because the sill of the valves was 7 m above the bed. Data on the efficiency of sediment sluicing during 1953 to 1958 are given in Table 4.15.

4.2.3.4 Density currents in the Guanting Reservoir. The Guanting Reservoir began detaining floods during the flood season of 1953, and it began impounding water in August 1955. During the detention period, i.e., before August 1955, the water level in the reservoir fluctuated within a rather wide range. There occurred many periods of reservoir emptying.

During the period of detention operation, the sluice gates were usually partly opened and during the impounding period the sluice gates were not opened; thereby detaining the flood waters, decreasing the flood discharge peak and permitting some outflow downstream of the dam. Alternatively, the gates were opened for a short interval of time to lower the pool level and to maintain a certain capacity for storing further flood waters. Under such conditions of reservoir operation, the sediment deposited amounted to 268.3×10^6 tons during the period of 1953 to 1957, corresponding 63.4 percent of the total inflowing silt discharge. The sediment released from the reservoir consisted of two parts, partly due to the erosion of the emptied reservoir bed and partly due to venting of density currents.

When the reservoir was operated in the impounding mode in 1956 to 1957, the silt released from the reservoir came entirely from vented density currents. The amount of sediment outflow was 8.3 percent of the total sediment inflow during this two-year period.

Insofar as the density currents are concerned, it is better to study the ratio of sediment outflow to the inflow during a period of an individual flood, based on the mechanism of density current movement, rather than the annual ratio of outflow to inflow of sediments.

The details of several such periods of density current are given in Table 4.15.

The longitudinal variations of the density currents in the Guanting Reservoir, the cross-sectional variations of velocity, and suspension concentration are depicted in Figures 4.40 and 4.41.

When the sluice gates are closed, a density current reaching the dam is backed up, forming a muddy pond.

From the data of field measurements and laboratory experiments, the condition of formation of a density current, as well as the conditions required to maintain the density current so that it will reach the dam and be vented out if the outlet is opened in time, have been analyzed (Fan 1962).

Table 4.15. Data on density currents vented out from reservoirs

Dam and Reservoir	Dam height (m)	Storage capacity	Annual runoff	Period yearly or of a flood	Silt discharge		Ratio of outflow to inflow of silt discharge	Reference
					Inflow	Outflow		
Iril Emda, Algeria	75	$160X10^6m^3$	$210X10^6m^3$	1953-1954	3,727,000 m^3	039,000 m3	0.25	Raud
				1954-1955	1,084,000	467,000	0.43	
				1955-1956	5,339,000	2,603,000	0.49	
				1957-1958		6,424,000	0.60	Duquennois
				1953-1954		1,000,000T	0.45	Duquennois
				1954-1955		600,000		
Lake Mead, U.S.A.		$38.4X10^9m^3$	$16X10^9m^3$	Mar, 30-April 17, 1935	$7.78X10^6T$	$1.79 X 10^6$	0.23	
				Aug.26-Sept. 9, 1935	$9.48 X 10^6$	$2.37 X 10^6$	0.25	
				Sept.27-Oct. 7, 1935	$8.35 X 10^6$	$3.27 X 10^6$	0.39	
				April 13-24 1936	$11.08 X 10^6$	$2.0 X 10^6$	0.18	
Nebeur Dam, Tunisia	65	$300X10^6m^3$	$180X10^6m^3$	May 1954 - May 1980	$4.9X10^6T/yr$	$3.5X10^6T/yr$	0.59-0.64	Abid
Fengjiashan Reservoir, China	77	$398X10^6m^3$	$485X10^6m^3$	Aug.6-8,1978	$459X10^3T$	$106X10^3T$	0.23	
				July 25-26, 1979	$1,180X10^3$	$767X10^3$	0.65	
Guanting Reservoir, China	45	$2,270X10^6m^3$	$1,400X10^6m^3$	July 2-5, 1954	$7.86X10^6T$	$2.7X10^6T$	0.34	
				July 4-25, 1954	$13.50X10^6$	$4.0X10^6$	0.30	
				July 28-29, 1954	$0.58X10^6$	$0.187X10^6$	0.32	
				Aug. 24-27, 1954	$5.30X10^6$	$1.06X10^6$	0.20	
				Sept. 5-6, 1954	$3.14X10^6$	$0.8X10^6$	0.20	
				June 26 - July 6, 1956	$20.5X10^6$	$4.56X10^6$	0.22	
				Aug. 1-3, 1956	$6.34X10^6$	$1.58X10^6$	0.25	
Sanmenxia Reservoir, China	106	$9,640X10^6m^3$	$43,200X10^6m^3$	July 2-8, 1961	$1.17X10^6T$	$0.014X10^6T$	0.012*	
				July 12-18, 1961	$1.09X10^6$	$0.061X10^6$	0.056*	
				July 21-28, 1961	$1.63X10^6$	$0.29X10^6$	0.18	
				Aug. 1-8, 1961	$1.7X10^6$	$0.30X10^6$	0.18	
				Aug. 10-28, 1961	$1.47X10^6$	$0.31X10^6$	0.21	
				Aug. 22-28, 1961	$0.81X10^6$	$0.069X10^6$	0.085+	
				Sept. 27-Oct. 2, 1961	$0.64X10^6$	$0.038X10^6$	0.06+	

* Density current accumulated in muddy pond formed by a submerged dam due to bank sliding.
+ Water level in reservoir is rising due to impounding at the end of flood season.

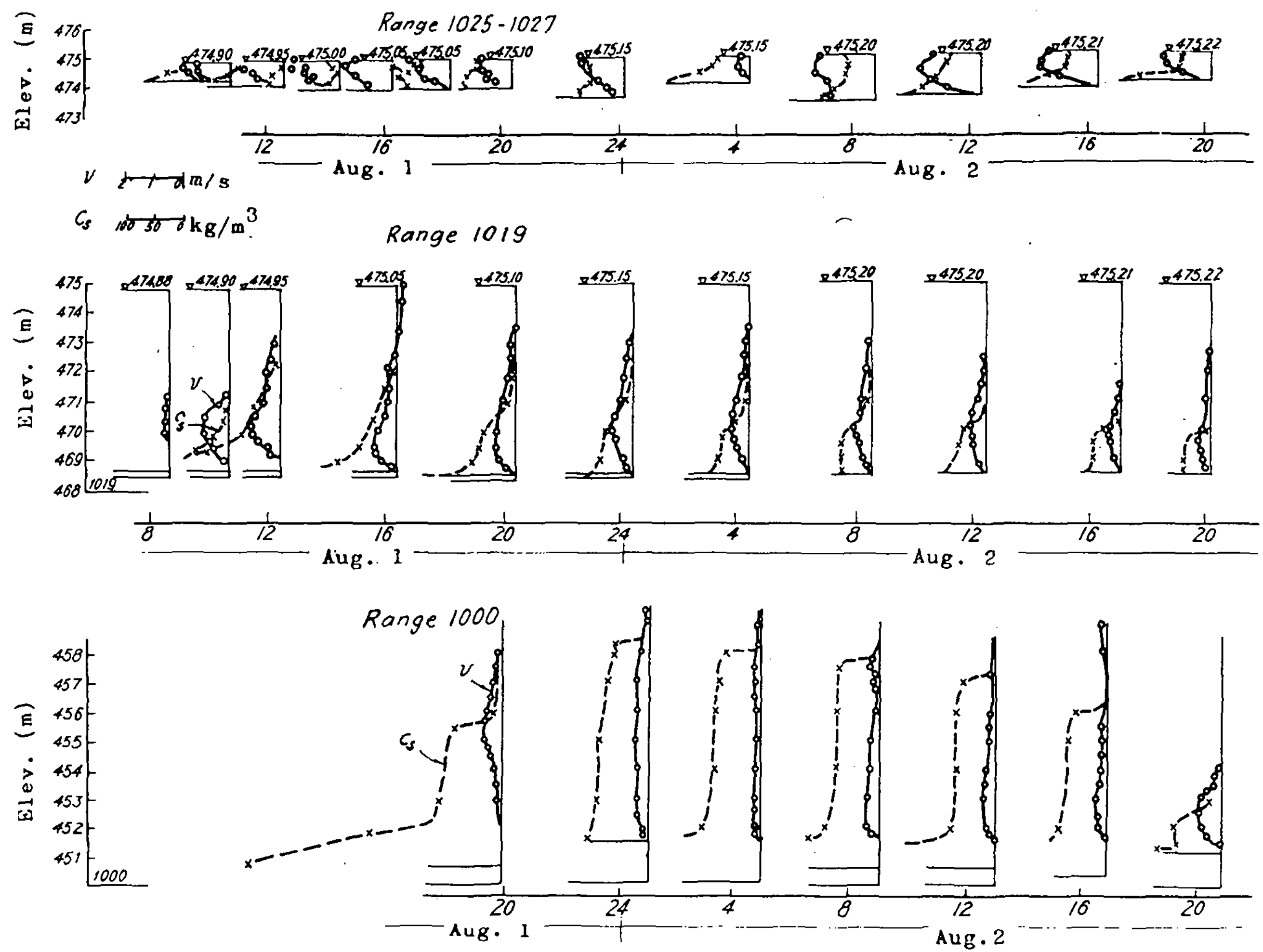


Figure 4.40. Density currents in Guanting Reservoir

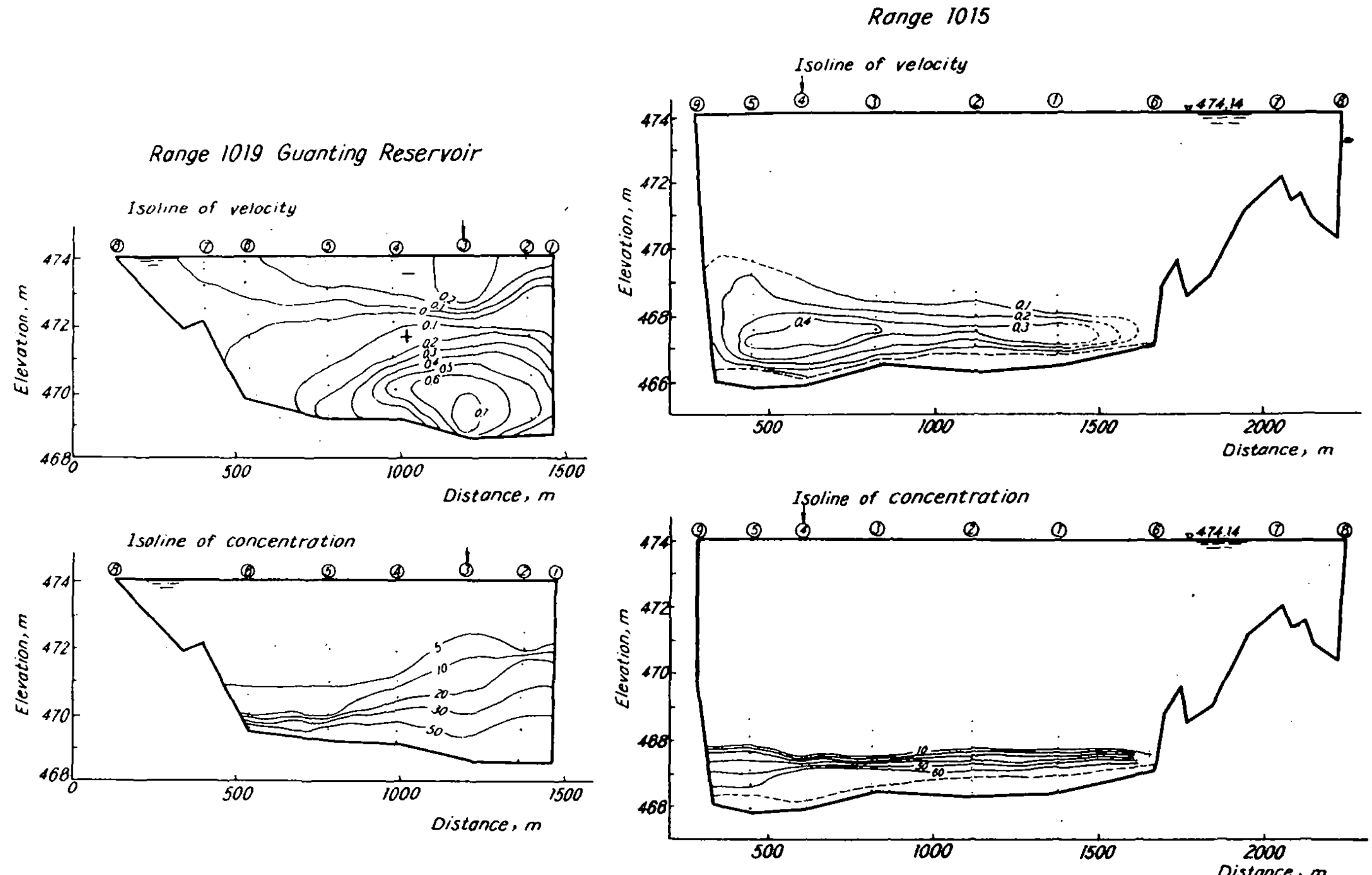


Figure 4.41 Density current measured on July 19, 1956, Guanting Reservoir

4.2.3.5 Density currents in the Sanmenxia Reservoir. After the dam was completed in September 1960, the reservoir began to impound water. Density current measurements were started in the flood season of 1961.

The hydrographs in Figure 4.42 show the inflowing and outflowing silt concentration. The profiles of velocity and silt concentration of the density current measured on August 16 through 18, 1961, are shown in Figure 2.7c.

In order to measure the unsteadiness of the density current, observations of the velocity and silt concentration profiles at various times and many sites were obtained. From these, the thickness of the density current may be obtained. Data from measurements made during the August 1961 floods are plotted in Figure 4.43. This reveals the unsteady density current which corresponds to the inflow variation at the Tongguan station, 114 km above the dam site (Figure 2.7a).

The density current may have backed up when it reached a submerged dam formed by land slipping into the reservoir, some 15 km from the dam site. Consequently, the density current is probably decreased at the outflow (see Table 4.15) during the period of July 2 through 18, 1961. If the submerged dam had not existed, the silt quantity transported by the density currents in the first two periods of flood would be increased, because there would have been no loss of density current above the submerged dam and the bottom slope would have been steeper (see Figure 4.44). This submerged dam has made the bottom slope milder for the later density currents and accordingly decreases their velocities, resulting in a decrease of the silt quantities in the vented density currents.

The ratio of the outflowing to inflowing silts of the density current for a flood period ranges from 0.18 to 0.21 in the Sanmenxia Reservoir under favorable conditions, i.e., no backwater in the density currents and no rising of pool water during flood periods.

4.2.3.6 Fengjiashan Reservoir. The Fengjiashan Reservoir, of 398 X $10^6 m^3$ storage capacity, was built in 1974 on a tributary of the Wei Ho River, which, in turn, is a tributary of the Yellow River, in northwest China. Tunnels on the right and left river banks and other sluices were provided for releasing the flood and density currents; also for draining or emptying the reservoir. Venting density currents to decrease the sediment retained in the reservoir was considered in the design stage.

The annual runoff is 485 X $10^6 m^3$, 44 percent of which occurs during the flood season. The sediment inflow amounts to 4.96 X 10^6 tons, 84 percent of which is carried during the flood season. The average sediment concentration is 9 kg/m^3, while the maximum observed value is 604 kg/m^3.

About 14 periods of density currents have been measured during the period from 1976 through 1980. The maximum sediment concentration of a density current reached as high as 676 kg/m^3. The ratio of sediment outflow to inflow discharge for each flood peak ranged from 23 to 65 percent according to the nature of the flood, length of reservoir, and sediment characteristics of the inflow.

The topographic features of the reservoir and the hydraulic structures for sluicing are favorable for venting the density currents in Fengjiashan Reservoir. The reservoir has a steep slope of the original river channel, fine sediment from the watershed area, a relatively short distance of backwater, and bottom sluices located just above the river bed. Other outlets at higher levels are also capable of flushing heavy discharge.

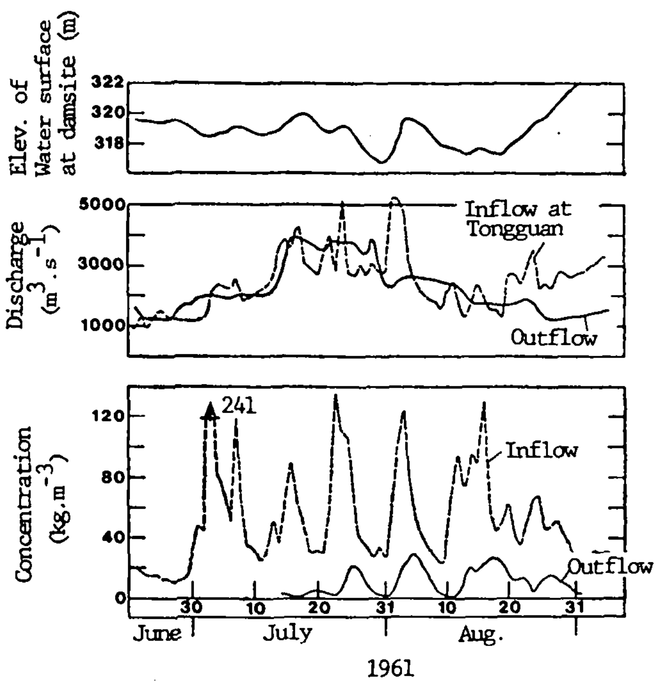


Figure 4.42. Density currents in Sanmenxia Reservoir. 1961

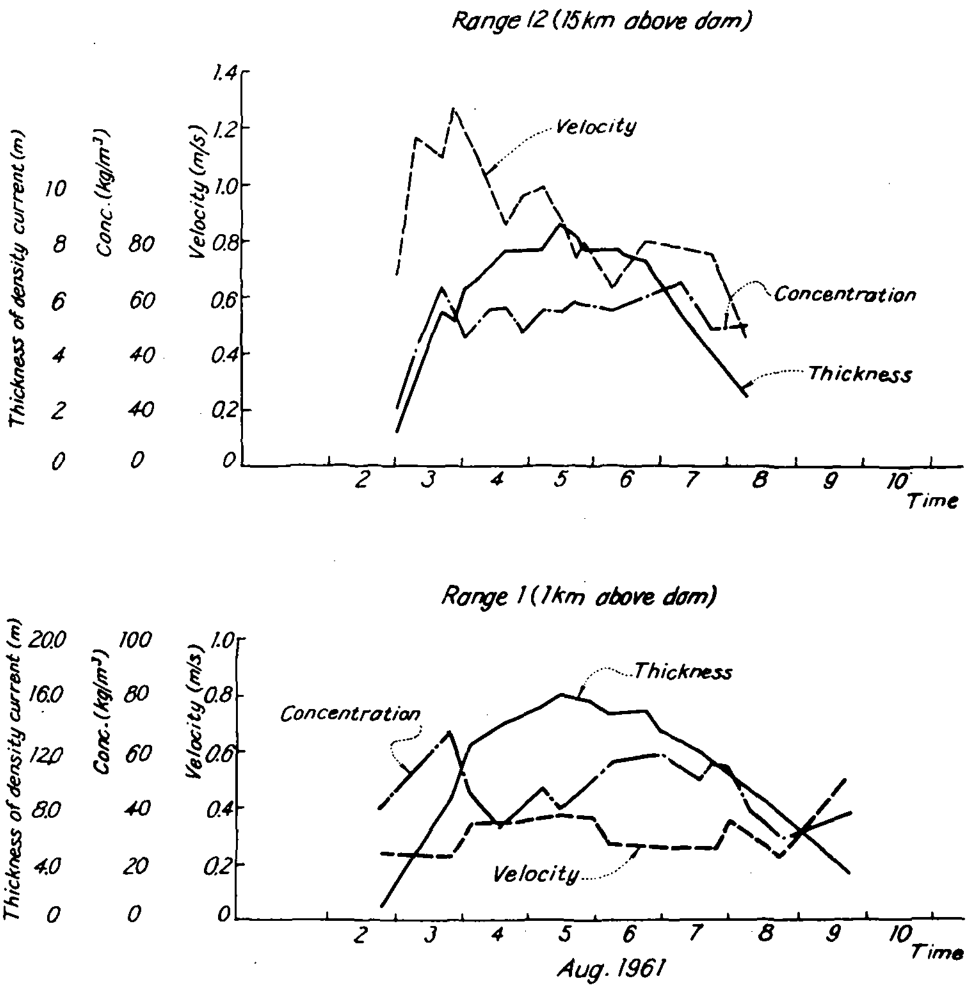


Figure 4.43. Unsteady density current measured in Sanmenxia Reservoir

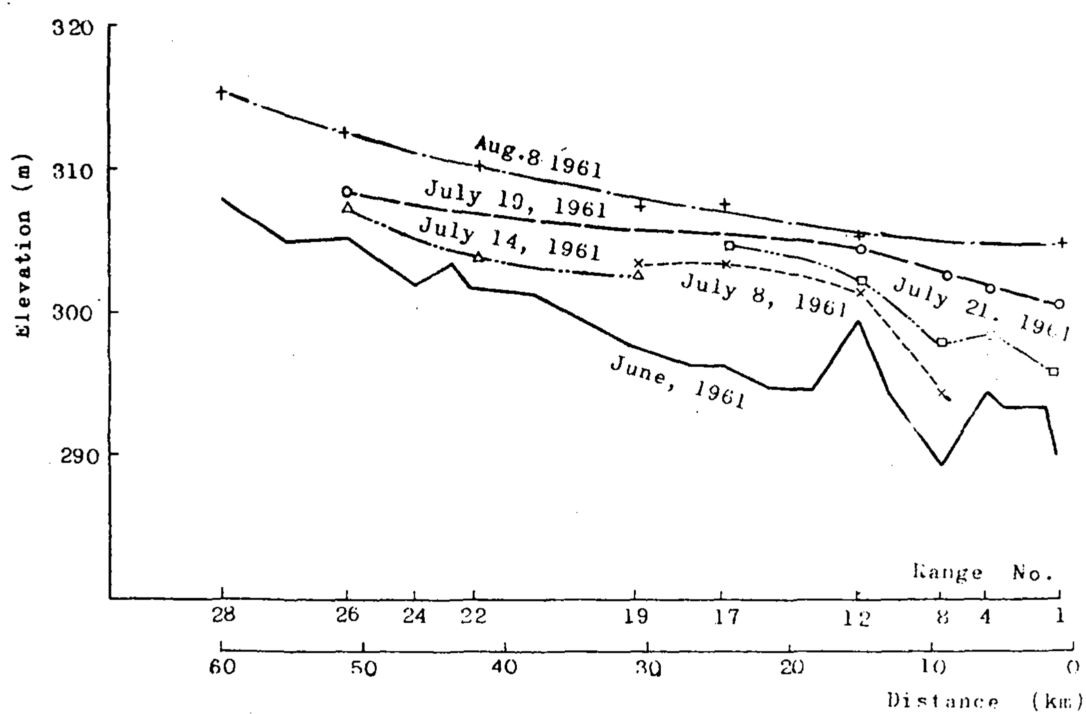


Figure 4.44. Interface of density currents measured in Sanmenxia Reservoir (Fan Jiahua)

Experience has shown that when the inflow discharge is greater than 50 $m^3.s^{-1}$ with a sediment concentration greater than 30 kg/m^3, a density current may be formed which will travel through the entire length of the reservoir to the dam. From the observed inflow and outflow sediment discharge, confirmed by discussion with local engineers, it appears that during the early years of operation there were occasions when the outlet gates were not opened by the time the density currents reached the front of the dam. Much more sediment could be evacuated from the reservoir by releasing the density current if the outlets could be controlled correctly.

4.2.3.7 The Nebeur Reservoir in Tunisia (Abid, 1980). The Nebeur Reservoir is located on the Mellègue Wadi in northwest Tunisia. The catchment area at the dam site is 10,300 km^2, 46 percent of which is bare, easily eroded terrain. The annual runoff is 180 X $10^6 m^3$.

The Nebeur dam is designed to regulate the course of the Oued Mellegue, the principal southern tributary of the River Medjerda. The water retained by the dam is destined to supply an irrigation network of 40,000 ha situated in the lower valley of the Medjerda. In addition, the head created by the dam is utilized for a hydroelectric power station producing 17,000,000 kwh p.a.

Two vents were fitted with Neyrpic valves with a capacity of 12.5 $m^3.s^{-1}$. A Bafour valve was fitted with a capacity of 1 $m^3.s^{-1}$.

The operation of venting a density current from the Nebeur Reservoir is conducted according to the variation in sediment concentration of the density current as it reaches the dam. When the density of the flow to be discharged approximates 1.08, the Bafour valve begins to work. As soon as the density exceeds 1.08, this valve is relieved by Neyrpic valves 1/4, 1/2, 3/4 or fully opened as necessary. When the density of flow falls to a value below 1.02, the venting operation stops.

During the period 1954 through 1980, the total amount of sediment discharged from the reservoir was 90.7 X 10^6 tons. The average ratio of outflow to inflow of sediment discharge, i.e., the average drawing-off efficiency is 59 to 64 percent.

4.2.3.8 Primary factors that affect the movement of density currents in reservoirs. An approximate method for computing the behavior of a density current has been developed by Fan (1962) who used data on the density currents of Lake Mead and the Guanting Reservoir.

The concentrations released by density currents depend on the topographic features of the reservoir (variation of width of the density current), the magnitude of incoming flood peak, incoming silt discharge and its sediment characteristics, the outlet elevation relative to the elevation of reservoir bottom, discharge capacity of outlets, flushing discharge, water level in the reservoir during the period of venting, and the length of the reservoir, etc. Density currents move along the original river channel during the construction of the dam and the first period of reservoir operation. Later the density currents move on a wider reservoir bottom after the original river channel has been filled. The unit discharge of density currents becomes smaller than that during the first period of operation.

Generally speaking, more sediments will be vented from short reservoirs with large incoming discharges, high density current concentrations, low and large outlets, and high outflow discharges.

Data on the ratio of outflowing sediment quantity in density currents to incoming silt quantity for a flood period are given in Table 4.16

Table 4.16. Summary of the measured ratio of density current outflow to inflow of sediment quantity for a flood period

Name of Reservoir	Length of reservoir (km)	Measured ratio of outflow to inflow of sediment quantity
Heisonlin	2	0.16 - 0.59
Fengjiashan	12 - 14	0.23 - 0.65
Guangting	4.8 - 8.5	0.19 - 0.34
Guangting	10.1 - 16.8	0.20 - 0.29
Sanmenxia Reservoir	80 (1961)	0.18 - 0.21
Lake Mead	128	0.18 - 0.39

A plot of the relationship between W_o/W_i and L, the length of the reservoir against the value of parameter $\bar{Q}_o/\bar{Q}_i$, where W_i, W_o are, respectively, the silt quantity of inflow and the silt discharge of outflow vented in form of density current, and $\bar{Q}_i$, $\bar{Q}_o$ are, respectively, the mean discharge of the inflow and outflow during the flood peak is given in Figure 4.45. The ratio W_o/W_i increases as the length of reservoir decreases and also as the ratio of $\bar{Q}_o/\bar{Q}_i$ increases.

4.3 BOTTOM OUTLET STRUCTURES

Whenever reservoir silting is likely to be a problem, the potentialities of bottom outlet release of sediment through a dam should be considered in the dam design; and, at the same time, the form of reservoir operation should also be taken into consideration. Of all the methods for sediment sluicing, the use of bottom outlets seems to be one of the most effective.

(1) Bottom outlets may be operated for undersluicing the flood, or draining the reservoir under emergency conditions when lowering the reservoir water level is urgently needed in a short period of time.

(2) Bottom outlets may be used for sluicing sediment by drawing down the water level in the reservoir, to release sediment deposits (silts, sands and gravels), which are eroded by the tractive force of the flow. Similarly, density currents may be vented from an impounding reservoir.

(3) When bottom outlets are located below the power intake, they may be useful in preventing the silt from entering the power intake, minimizing the possibility of wear occurring in water turbines.

(A) As far as the bottom outlets are concerned, the Spanish undersluicing is the oldest sediment sluicing device. It consists of an opening near the original river bed or the base of the dam. Flood flushing by this method has been effective in maintaining the efficiency of some reservoirs for a hundred years. One of the famous examples is the old Aswan Dam located on the Nile near Haifa,

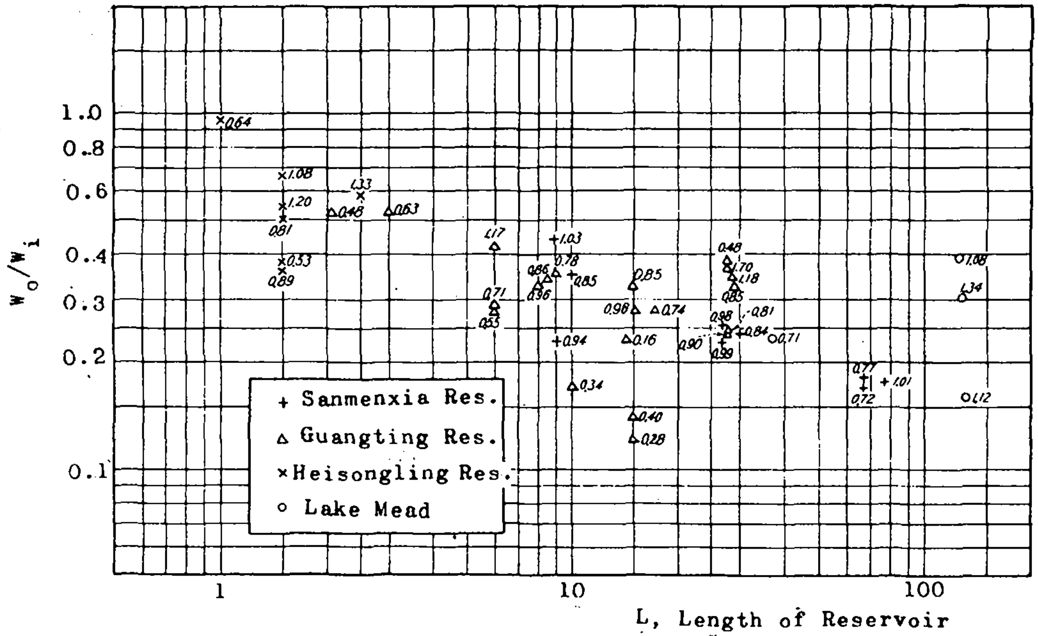


Figure 4.45. Relationship between W_o/W_i and L with a parameter $\bar{Q}_o/\bar{Q}_i$ (Fan Jiahua)

Egypt. The principle was applied of allowing the sediment-laden water of floods to pass through the reservoir basin without appreciable diminution of velocity. One hundred and eighty sluices, built into the dam in 4 groups at different levels, were together capable of passing a maximum flood of 14,200 m^3/s.

This is an example of a dam which was designed specifically with bottom outlets, capable of discharging a heavy flood with complete drawdown of pool level.

(B) Another example is the Gebidem Reservoir on the Massa River in Switzerland (Swiss National Committee on Large Dams, 1982). This reservoir is regularly flushed once a year when the river flow is sufficiently high (normally at the beginning of June). The narrow valley bottom considerably increases the effectiveness of flushing.

Model tests and field measurements have been carried out to investigate the effectiveness of the bottom outlets.

The 122-m high Gebidem Dam, completed in 1968, has a storage capacity of $9 \times 10^6 m^3$ which is used for power generation. The annual runoff of $420 \times 10^6 m^3$ contains about 500,000 m^3 of sediment of which 130,000 m^3 is bed load of sand and gravel. One of the important problems to be solved was sedimentation in a reservoir of comparatively small storage capacity which receives a considerable amount of sediment discharge. Model tests were undertaken to confirm the possibility of the sediment flushing by completed drawdown of the pool level.

Model tests have been carried out on two modes of flushing:

- Orifice flow (without interruption of the power generation, the water level is maintained a little higher than the minimum water level of exploitation), and
- Free-surface flow (total cessation of generation) during flushing through the bottom outlet, the results are tabulated in Table 4.17.

Table 4.17. Data of model test, Gebidem Reservoir

	Orifice flushing	Free-surface flushing
No. of flushing per year	15	1
Ratio between water volume and sediment wt. of outflow (m^3/t)	14	32
Volume of water needed ($10^6 m^3$)	1.68	3.84
Mean discharge of flushing ($m^3.s^{-1}$)	115	30
Duration of flushing (hr)	1	35

The bottom outlet of the dam was specially designed for annual flushing of the sediment deposits in the reservoir. Results of prototype measurement since 1969 are shown in Table 4.18.

The results of the model tests of the the flushing procedure proved satisfactory when compared with the data of measurements in situ from 1969 to 1978. The efficiency of flushing, represented by the water-sediment ratio, was about twice as high as predicted by the model test. The operation allows a mean

Table 4.18. Flushing data, 1969-1977, Gebidem Reservoir

Year	Vol. of water used for flushing (m^3)	Vol. of sediments evacuated (m^3)	Vol. of water / Vol. of sediment	Duration of flushing (hr)	Max. concentration of Viège (g/l)
1969	1,100,000	49,000	22.4	46	10.5
1970	1,650,000	80,000	20.6	44	12.5
1971	2,130,000	130,000	16.4	47	30
1972	2,087,000	104,000	20.1	42	30
1973	1,837,300	110,200	16.8	41	30
1974	2,471,900	148,000	16.7	43	60
1975	1,060,000	64,000	16.5	48.75	27.5
1976	2,271,000	136,000	16.7	47	33.7
1977	1,930,000	115,800	16.7	46	26.25

sediment washout of 100,000 m^3, which more or less corresponds to the annual sediment deposits. The last column in Table 4.18 indicates the sediment concentration flowing into the Rhone at Viege, downstream of the dam. A photograph of the reservoir at the end of the 1976 flushing is shown in Figure 4.46. The plan of bottom outlet of the Gebidem Reservoir is given in Figure 4.47.

(C) A third kind of bottom outlet is that installed with valves for density current withdrawal. The following cases may be noted (Groupe de Travail du Comite Français des Grands Barrages, 1982).

In Tunisia, the Nebeur Dam on the Oued Mellegue contains one Bafour valve and two Neyrpic valves. The plan of this structure is shown in Figure 4.48 and a general view of the desilting sluice is shown in Figure 4.49.

The Bebhana and Bir M'Cherga Dams have 4 conduits, 400 mm in diameter and 8 small valves are installed in these two dams.

In Algeria, the Es Saada on the Oued Mina has 4 conduits 400 mm in diameter and 8 small valves.

Iril Emda Dam has 8 conduits 400 mm in diameter (bypass through the main gates).

In Morocco, the Youssef Ben Tachfine Dam and Tleta Dam has a bypass, 400 mm in diameter, as a bottom outlet.

The operation of the valves for venting density currents has proved satisfactory. For example, some 60 percent of the inflowing sediment is flushed every year from the Nebeur Reservoir (Abid, 1980).

Figure 4.46. Photograph of reservoir at the end of the 1976 flushing, Gebidem Dam, Switzerland (After Dawans et al., 1982)

(D) In some cases, the bottom outlet originally provided has become obstructed completely by sediments probably as a result of incorrect operation of the reservoir. Consequently, a new outlet is opened for the purpose of sediment flushing, or the obstructed outlet is reopened.

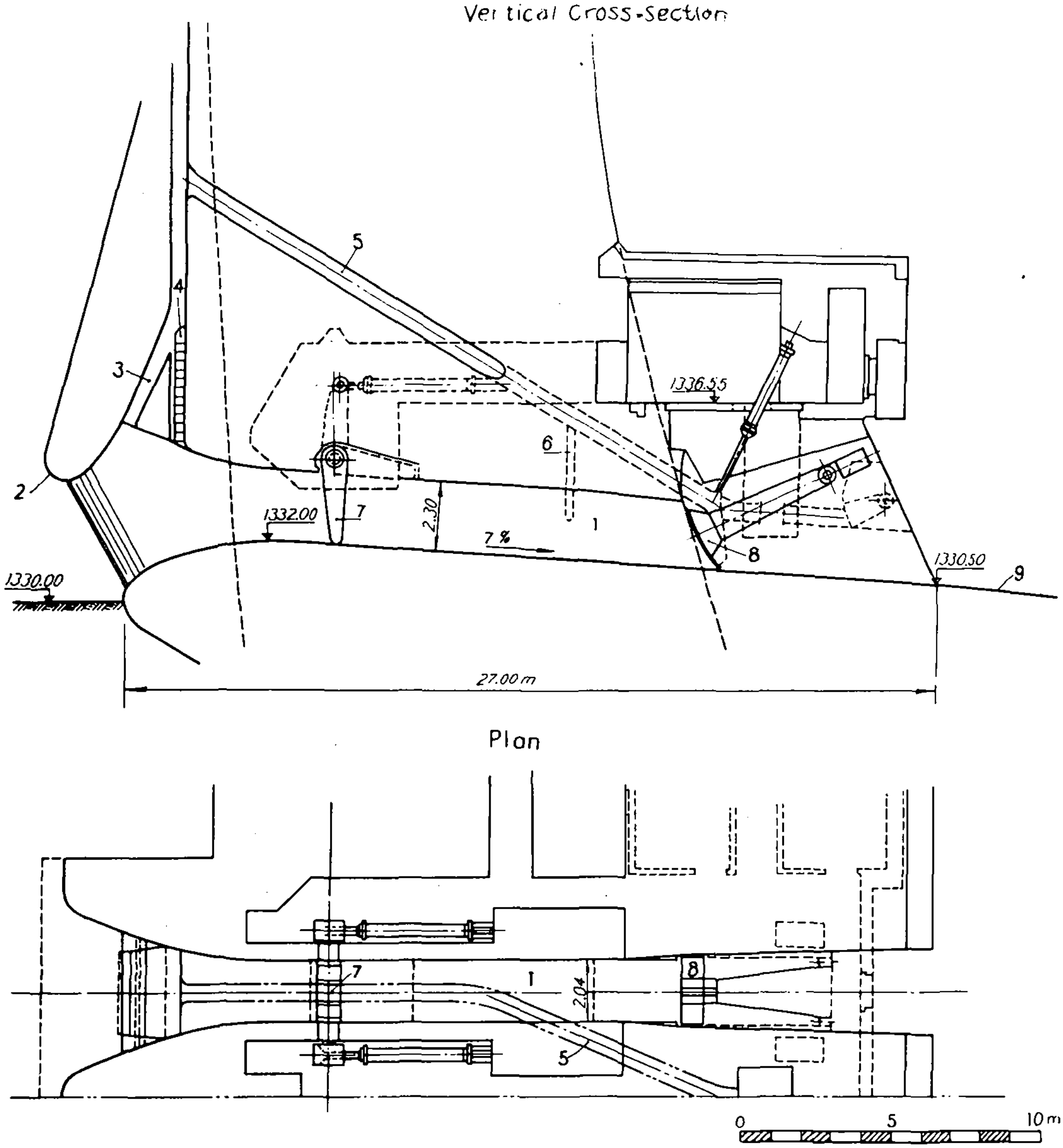


Figure 4.47. Vertical and horizontal section of the bottom outlet through the Gebidem Dam (After Dawans et al., 1982)

1. Steel-lined bottom outlet
2. Guard
3. Jetting pipe
4. Bulkhead gate
5. Compensation water pipe (ϕ60cm)
6. Bypass
7. Check valve
8. Radial Gate
9. Downstream apron

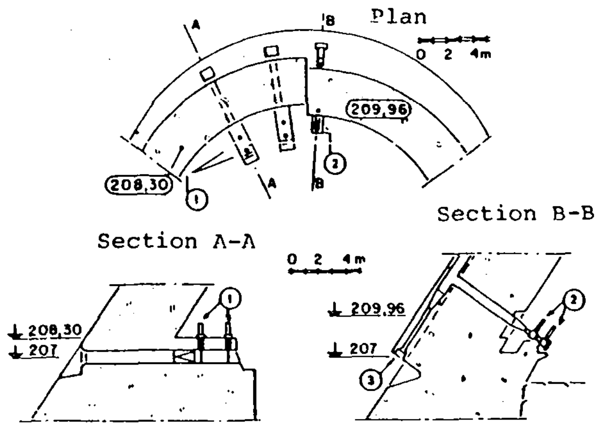


Figure 4.48. Valves for turbidity current withdrawal at Nebeur Dam (After Groupe de Travail du Comité Français des Grands Barrages, 1982)

1. Neyripc valve
2. Bafour valve
3. Screen

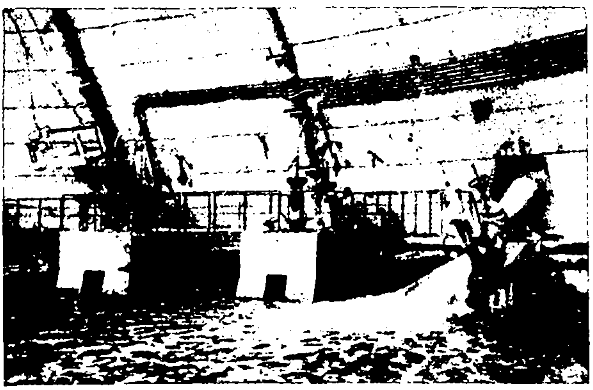


Figure 4.49. General view of the desilting sluice gates in the Nebeur Dam, Tunisia. On the left, the "Neyrpic" sluice gates, and on the right the silt extractor. (After A. de Montmarin, 1955)

New bottom outlets have been constructed for sediment release at the Chambon and Sautet Reservoirs in France where there has been severe reservoir sedimentation (Berthier et al., 1970).

The Chambon gravity dam, completed in 1935, is 136 m high, and its reservoir has a storage capacity of 56 X $10^6 m^3$. Bottom outlets with trash screens were provided in the original design. In 1955, however, it was found that they were entirely blocked by sediment and by 1959 the mud level had risen by 12 m, endangering the safety of the dam. During the reconstruction, the sediment deposits behind the dam and in the original outlet were removed and the diversion tunnel, used during the original construction, was reopened. The tunnel was 15 m higher than the original bottom outlet. Construction of a new bottom outlet was completed in December 1962 (Figures 4.50 and 4.51).

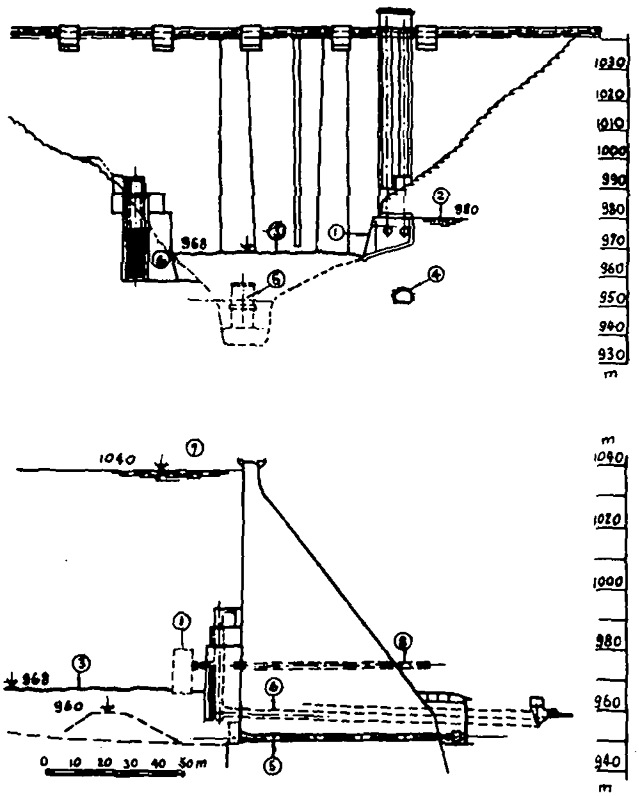


Figure 4.50. Chambon Dam - Construction of new bottom outlet. (After Berthier et al., 1970)

1. Intake
2. Min. operating level
3. Elevation of silt (before works)
4. Diversion tunnel
5. Old bottom outlet
6. New bottom outlet
7. Normal reservoir level
8. Adduction gallery

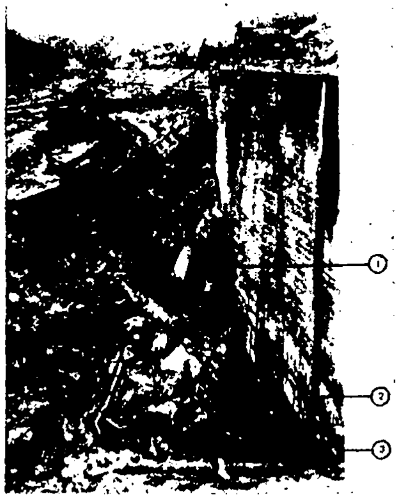


Figure 4.51. The new bottom outlet of Chambon Dam (After A. Berthier et al., 1970)

1. New bottom outlet
2. Old bottom outlet
3. Intake

The second example of installing a new outlet is at the Sautet Dam which is 115 m high. The reservoir behind the dam has 100 X $10^6 m^3$ storage capacity and started impounding in 1935. Two sluices were designed at different elevations, the first was a bottom outlet, the original diversion tunnel; the second was an intermediate outlet passing through the dam, 25 m above the bottom outlet. The bottom outlet was operated until 1938, when it was almost completely blocked by the sediments. The depositional surface rose to reach the intermediate outlet which became obstructed in 1961. The thickness of the deposits entrapped above the dam reached up to 50 m, only 15 m lower than the power intake; therefore, it was decided to open a new sluice. The sluice was located 5 m below the intake and was completed in 1962 through 1963 (Figures 4.52 and 4.53).

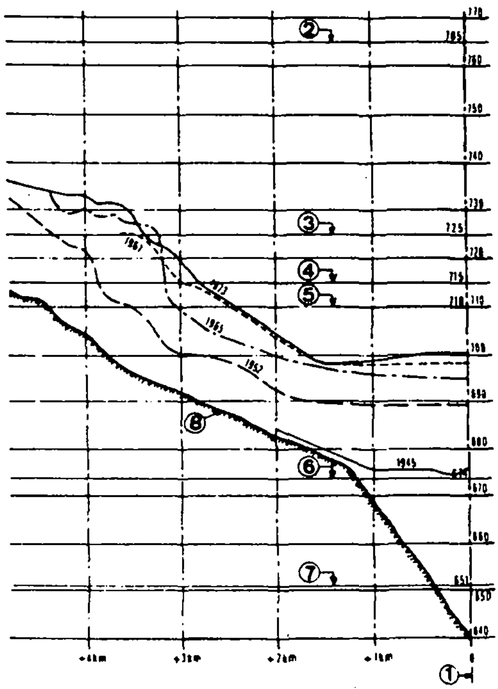


Figure 4.52. Sautet Reservoir - changes in longitudinal profile between 1935 and 1973 (After Groupe de Travail du Comité Français des Grands Barrages, 1976)

1. Dam
2. Maximum reservoir level
3. Minimum operating level
4. Intake sill level
5. New bottom outlet sill level
6. Former intermediate outlet sill level (abandoned)
7. Former bottom outlet sill level (abandoned)
8. Reservoir bottom profile before impounding in 1935

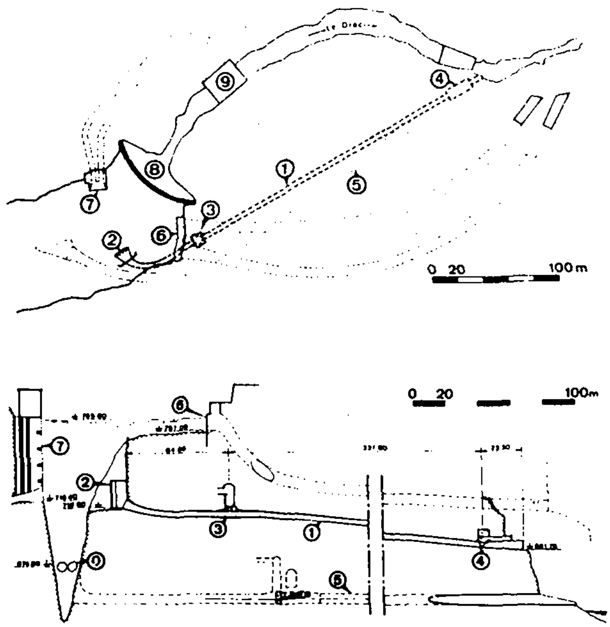


Figure 4.53. Sautet Dam - new deep outlet (After Groupe de Travail du Comité Français des Grands Barrages, 1976)

0. Intermediate outlets
1. New 3 m dia. outlet
2. Outlet entrance
3. Upstream stoplogs
4. Downstream regulating gate
5. Former bottom outlet
6. Surface spillways
7. Power intake
8. Dam
9. Power station

The new bottom outlets installed in these two reservoirs were used to vent density currents during floods.

It is reported further that the Steeg Dam on the Oued Fodda in Algeria had a bottom outlet which became obstructed by sediments after several years of operation. Orifices located below the sediment surface level behind the dam were opened to vent the deposits from density currents accumulated previously (Figures 4.54 and 4.55) (Thevenin, 1960a).

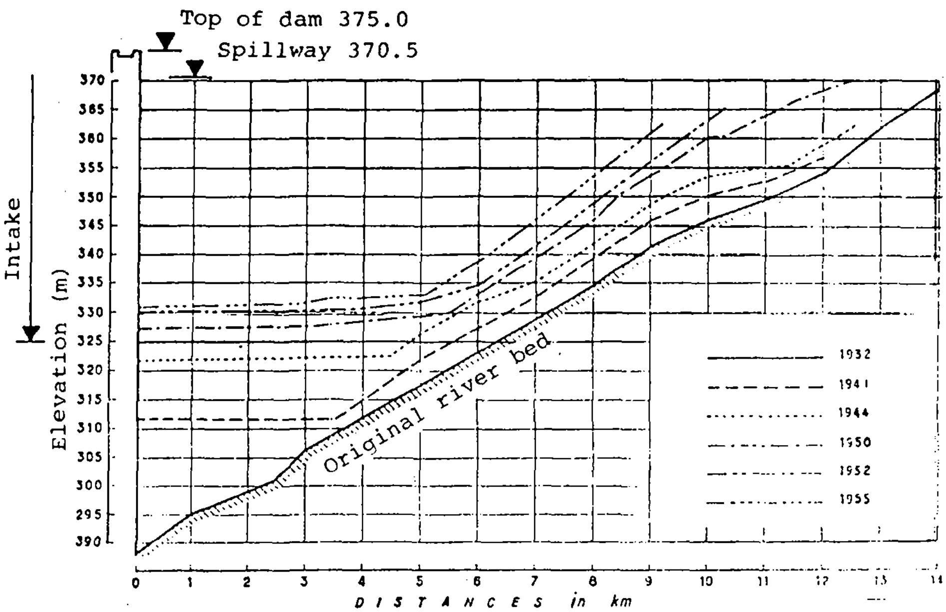


Figure 4.54. Steeg Dam on the Oued Fodda - profiles of sediment deposition (After Thévenin, 1960)

Figure 4.55. Outflow of mud vented from the reservoir on the Oued Fodda, Alegeria (After Thévenin, 1960)

(E) A bottom outlet is provided for sediment withdrawal to maintain the local zone around the power intake free from sediment deposits so that no course sediment can enter the intake. One example is the Lake of Grimsel, the oldest storage reservoir in Switzerland (Swiss National Committee on Large dam, 1982). This 100 X $10^6 m^3$ reservoir accumulated 1.65 X $10^6 m^3$ of sediment within a period of 40 years, a volume which is less than 2 percent of the storage capacity. On the occasion of a completed drawdown in 1973, sediment deposition had not only reached the dam site, but had risen to the elevation of the power intake. The old power intake was transformed into a scour outlet to provde a local free-flushed zone around the new intake which was built immediately above the old one (Figures 4.56 and 4.57).

(F) The Sanmenxia Reservoir in China is an example of dam reconstruction, including the building of two new side tunnels, and transforming and rebuilding 4 penstocks for discharging flood water. Eight of 12 bottom sluices which had been plugged after the completion of the dam, were reopened for flood discharging and sediment release.

After the completion of the dam, the reservoir began to impound water in the period September 1960 through March 1962. Severe sedimentation occurred. The mode of operation was changed to include flood flushing during the flood season, but deposition of sediment continued owing to insufficient outlet discharge capacity. Reconstruction of the dam was required in order to increase the discharge capacity.

This is an example of a dam for which no bottom outlet was provided in the design; then as a result of severe sediment siltation, new building work of bottom outlets, such as tunnels, etc., had to be undertaken in order to release sediment.

It is obvious that it is costly to reconstruct the bottom outlets of existing dams, and it is vital that provision for these outlets be considered in the design stage. It is an important problem for dam building in arid and semiarid regions, where the sedimentation in reservoirs is usually serious.

Insofar as the management of the reservoir is concerned, the importance of timing in the operation of the outlet for releasing sediment during the flood season should not be underestimated. For example, in venting a density current, the outlet or valves must be opened neither too early, nor too late for effective flushing of the sediment (Groupe de Travail du Comité Français des Grands Barrages, 1982; Fan, 1962).

4.4 RECOVERY OF STORAGE

4.4.1 Flushing of Deposited Sediment

Reservoir-emptying operations may be used periodically for small reservoirs where the storage capacity could not be maintained for beneficial use after a period of several years of operation. Since a great part of the useful storage capacity in a small reservoir is located near the dam site, the sediment deposits may be removed by flood flow if the outlet gates are left open for a period of time. The channel, thus scoured out in the deposits, becomes a part of the storage capacity.

Emptying and flushing operations may be used in reservoirs where a balance between deposition and erosion cannot be obtained by flushing sediment during the flood and storing clearer water during the nonflood seasons.

An example of a reservoir operated by emptying and flushing is the Hengshan Reservoir, a small gorge type reservoir, 1 km in length, with a storage capacity of 13.3 X $10^6 m^3$. The 69 m high dam has a small bottom outlet for a discharge of 17 $m^3.s^{-1}$, 2.6 m above the original river bottom, and an outlet for flood flushing placed at 14.5 m above the river bed, capable of passing a maximum discharge of 1260 $m^3.s^{-1}$. The reservoir is used for flood control and irrigation.

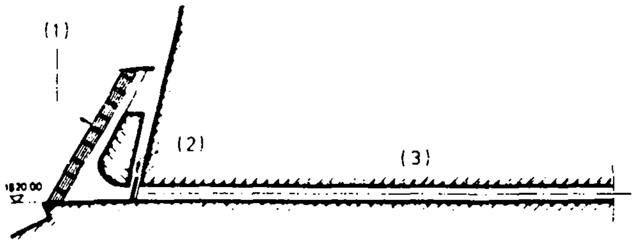


Figure 4.56. Grimsel Reservoir - power intake before transformation (After Swiss National Committee on Large Dam, 1982)

1. Intake rake
2. Roller gate
3. Old pressure gallery

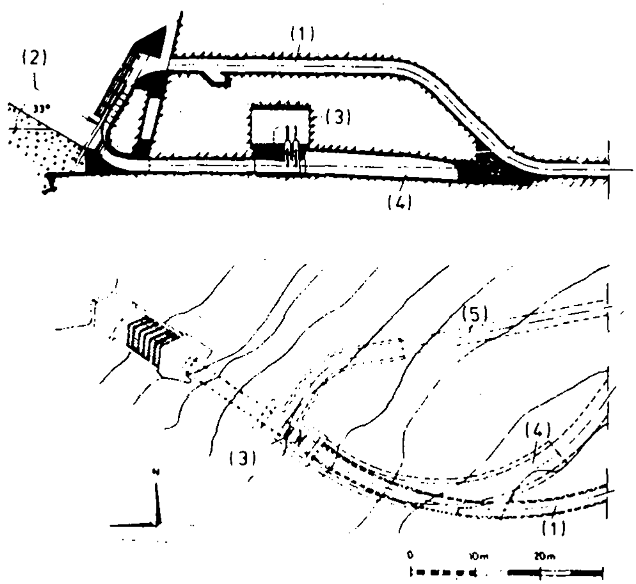


Figure 4.57. Grimsel Reservoir - power intake after transformation (After Swiss National Committee on Large Dams, 1982)

1. New pressure gallery
2. Freshwater duct
3. Gate chamber
4. Scour tunnel
5. Access gallery

After the first eight years of reservoir operation, 1966 through 1973, a volume of 3.19 X $10^6 m^3$ of sediment had accumulated. The height of the deposits behind the dam reached 27 m. The reservoir was emptied and flushed for 37 days in 1974 and a storage capacity of 800 X $10^3 m^3$ was recovered. After that, the reservoir was impounded until June of 1979, by which time the main channel that had been eroded in 1974 had been completely filled. A second emptying and flushing period which lasted for 52 days of the flood season of 1979 enabled 1.03 X $10^6 m^3$ of storage to be regained. The volume of deposits in the reservoir was reduced to 2.62 X $10^6 m^3$.

Longitudinal profiles obtained before and after emptying and flushing on each occasion are given in Figure 4.58. Periodical emptying and flushing every few years recovers useful storage capacity by removing flood plain deposits and forming a channel in the reservoir. Photos showing the emptying of the Hengshan Reservoir in 1982 are shown in Figures 4.59 and 4.60.

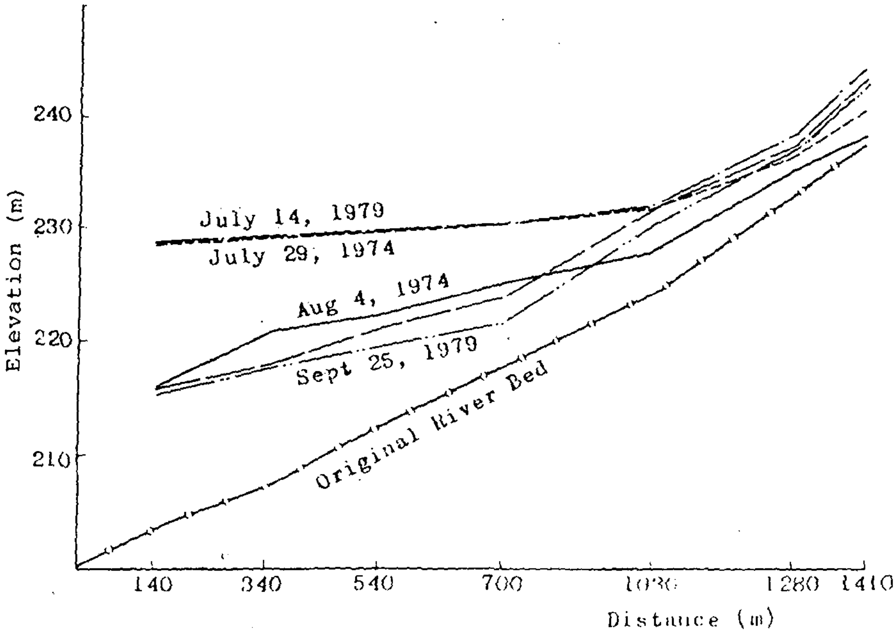


Figure 4.58. Profiles before and after emptying and flushing, Hengshan Reservoir

Field measurements show that the flushing in this emptied reservoir was in the form of strong retrogressive erosion. The outflow concentrations reached about 1000 kg/m^3, irrespective of the magnitude of discharge. This level was maintained for about 100 minutes in all cases, but in 1974, continued for over

Figure 4.59. Reservoir emptied three days after the emptying of the Hengshan Reservoir, China, in 1982. The eroded channel appeared at the initial stage under a discharge of 0.2 $m^3.s^{-1}$ (From Shanxi Institute of Hydraulic Research, China)

Figure 4.60. Eroded channel 40-110 m wide at the end of the 1982 flushing, Hengshan Reservoir, China. The water depth above the dam was about 25 m when the reservoir was impounding (From Shanxi Institute of Hydraulic Research, China)

1000 minutes, as shown in Figure 4.61. Maximum concentrations measured were 1240 kg/m^3 in 1975, and 1300 kg/m^3 in 1979. The concentration of outflow normally became smaller after a period of 2000 minutes.

During emptying and flushing, strong retrogressive erosion occurred as a result of lowering the water level in the reservoir. A channel was rapidly formed in the flood plain deposits, and this deepened continuously and extended upstream. Within 350 m of the dam, mud on the flood plain surface slid gradually into the channel, lowering the level of flood plain deposits. A transverse slope of 5.5 percent was measured at a cross section of 150 m from the dam site. The deposited fine sediments (D_{50} = 0.02 mm) had a high water content and were capable of sliding into the channel where they were easily eroded and released from the reservoir at high concentrations (800 to 1300 kg/m^3).

Between 350 and 800 m from the dam site, the sediment deposits were coarser. Slope slides of the mud deposits were also observed, but some small steps existed on the side slopes of the transverse cross sections due to the coarseness of the deposits on the flood plain.

In the reaches remote from the dam site, the cross section eroded was approximately rectangular in form, because the channel was broadened by the erosion at the foot of the banks, which led the coarse flood plain deposits to collapse vertically into the channel.

How to select the time for emptying and flushing and how to predict the instant and duration of flushing for evacuating the sediments are most important problems in reservoir operation.

Experience in the Hengshan Reservoir suggests that the efficiency of flushing was high when the main channel, eroded in a previous flushing process, had been silted by sediments during a period of several (4 or 5) years.

Still greater recovery of capacity could be achieved if the reservoir were emptied before the arrival of the flood, so that the flood waters could exert their strongest erosive force on the unconsolidated deposits, which had not yet consolidated after emptying of the reservoir. The period of flushing should be restricted to the flood season, so that no further serious deposition would occur after the flushing period.

4.4.2 Dredging

Dredging to remove sediment from the reservoir is undertaken where

(a) flushing is not successful,

(b) the building of a bypass is impossible,

(c) the drawdown of the pool level for flushing is not allowed for the sake of saving water,

(d) the dam is irreplaceable with no possibility of further raising the dam height, or

(e) the energy consumed in flushing by lowering pool level or emptying the reservoir is uneconomic for reducing the rate of silting in the reservoir.

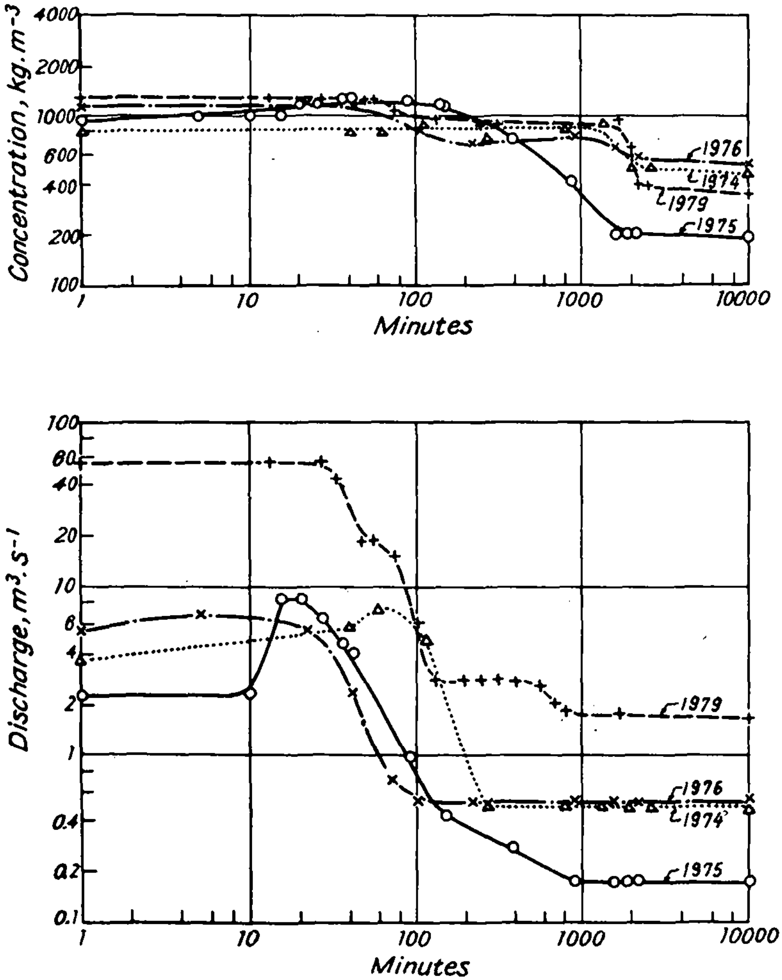


Figure 4.61. Outflow concentration from Hengshan Reservoir during emptying and flushing

Generally speaking, dredging is an expensive means of restoring the storage capacity of a reservoir unless the deposits removed can be used for the beneficial purposes. Some coarse sediments dredged may be used as construction material.

Dredging in reservoirs is performed under different conditions as follows:

(1) To recover the storage capacity of small-sized reservoirs, small compensation basins, gravel retention basins; or to partially recover the capacity of medium sized reservoirs.

In Algeria (Bellouni, 1980; Belachir, 1980), dredging is undertaken during the irrigation periods to regain storage capacity. The dredger *Lucien Demay* is a suction and force dredger with rotating cutter head, whose theoretical efficiency is that to dredge 1 m^3 of silt in situ (density approximately 1.6) requiring approximately 5 m^3 of clear water. The silt volume dredged per month is estimated to be about 340,000 m^3. When plugging occurs in the suction and force pipes, additional water is required resulting in a consumption of 9 volumes of water per volume of silt.

Dredging undertaken by the *Lucien Demay* between 1957 and 1968 may be tabulated below:

Year	Dam	Silt dredged ($10^6 m^3$)
1958 - 1961	Cheurfas	10
1962 - 1964	Sig	1
1965 - 1966	Fergoug	3
Nov. 10, 1967 - April 30, 1968	Hamiz	1.2

During a period of 5.5 months of the dredging operation in the Hamiz Reservoir, 2300 hours of effective pumping where recorded, while 730 hours of stoppage occurred during the operation. According to Bellouni, the main causes of stoppage were mechanical incidents and pipe-line incidents, "plugging" in pipes, changes of location, and lengthening and shortening of the floating pipe. The phenomenon of "plugging" caused 25 percent of the stoppage. It results because lumps of cut silt become meshed together with foreign bodies, such as tree stumps in the suction pipe. In order to clear it, it is necessary to back the suction pipe slightly to admit clear water to separate the lumps. Plugging may also occur in the force pipe as a result of the silt lumps not diluted in water.

The dredging program for desilting the Algerian reservoirs generally involves excavation from the downstream to the upstream to open a channel in the deposits and, thereby, to facilitate the movement of density currents toward the bottom outlets. These density currents have large unit discharges and increased velocities in the new submerged channel.

In the compensating reservoir of Palagnedra (Leichti et al., 1970), in Switzerland, a floating dredger is equipped to evacuate in depths of up to 50 m, the finest silt materials accumulated near the water intake. The materials sucked in are pushed through a floating pipeline to the dam, then through it, and later discharged down stream (Figures 4.62a and 4.62b).

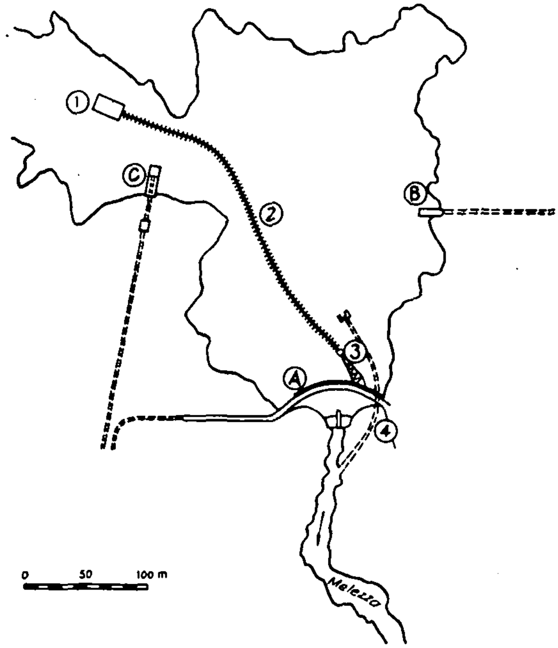


Figure 4.62a. Lower part of the basin near the dam, with floating dredger (After W. Liechti, 1970)

A. Dam
B. Outlet work of the headwater tunnel
C. Water intake and pressure tunnel
1. Floating dredger
2. Floating pipeline
3. Mobile element linked to dam
4. Pipeline through the dam

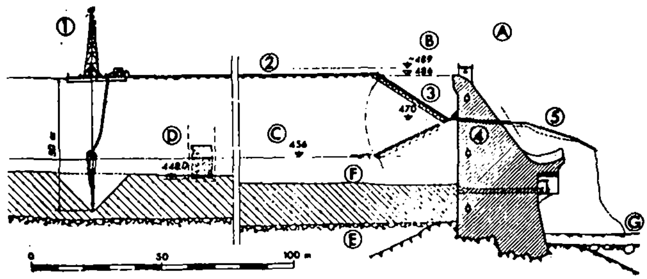


Figure 4.62b. Longitudinal section through lower part of the basin with floating dredger (After W. Liechti, 1970)

A. Dam, section through main axis
B. Maximum storage level without overflow, 486 m a.s.1.
C. Minimum storage level 456 m a.s.1
D. Water intake of Verbano pressure tunnel, sill elevation 448 m a.s.1
E. Old river bed
F. Elevation of the silting-up in June 1967
G. Downstream elevation of Melezza River, approximately 426 m a.s.1
1. Floating dredger, maximum extracting depth 50 m
2. Floating pipeline, length from 280 to 350 m, diameter 350 mm
3. Mobile element linked to the dam, length 32 m
4. Pipeline through the dam, length 20.5 m, diameter of the drilled hole 490 mm
5. Security valves and discharge pipe with outlet cone

(2) To clear the deposits in the backwater ponds of a chain of power stations, to lower the flood level in the river channel, or to maintain a necessary navigation depth in the backwater sedimentation reach of the reservoir.

In Austria, the development plan of hydroelectric power involving the building of a chain of power stations on the Austrian stretch of the Danube River was started during the fifties. The Ybbs-Persenbeug power station was the first put into operation in 1957. The reservoir capacity is 76 X $10^6 m^3$ with the active storage accounting for 44 X $10^6 m^3$, and the structure provides a mean head of 11 m.

Bed load settles close the upstream limit of the backwater pond, causing narrowing and shallowing of parts of the river bed and impeding navigation and river flow. Reduced flow sections entail raising of the water level and a higher flood risk. During the first four years of observation 800,000 m^3 of gravel was deposited in the upstream part of the backwater pond. These gravel deposits are periodially removed by dredging.

The bed load transport had been reduced to a large extent by the completion of the upstream power station of Wallsee-Mitterkirchen in 1968. It has an average head of 11 m and a storage capacity of 54 X $10^6 m^3$, with the active storage accounting for 29 X $10^6 m^3$. Considerable amounts of bed load were deposited, especially during the years with floods (about 300,000 m^3 in 1970, about 400,000 m^3 in 1975). These deposits were removed by dredging to prevent raising of the flood levels and maintain the clear height of 8 m required for navigation below the railway bridge.

Kobilka and Hauch (1982) have estimated that the cost of gravel dredging in the upper backwater zone is reasonable from an economic point of view. On the other hand, man-made overdeepening of the tailwater by dredging is considered the least costly way of increasing the head utilized for power generation. Moreover, the gravel removed is used for construction work elsewhere.

(3) In the case of deposits (gravels) that may be used as aggregates for concrete, the dredging method is not costly. There are many reservoirs in Japan where the deposits dredged out from the reservoir are used as concrete aggregates. Three reservoirs (Akiba, Sakuma and Miwa) fall into this category (Nose, 1982, Okada et al., 1982; Shiozawa, 1974; Murakami, 1979).

The Akiba Dam, 84 m high, built on the Tenryu River for the purpose of power generation, was completed in 1958. The total storage capacity is 34 X $10^6 m^3$ and the catchment area at the dam site is 4,490 km^2.

An annual removal of 400,000 m^3 of sediment is necessary to maintain the level of the original riverbed.

Sediments deposited in the 4.5 km nearest the dam consist of fine clay and silt, some 150,000 m^3 of which material are removed annually by pump dredger and piled just upstream of the dam, to be released with the flood discharges.

Sediments deposited in the region 4.5 km to 8.2 km from the dam are well graded, and are suitable for use as concrete aggregates. The volume of these sediments excavated annually is 200,000 m^3.

Sediments upstream of 8.2 km from the dam are gravels, some 50,000 m3 of which are excavated annually by shovel-type excavators. Some of this is used as concrete aggragate and some is wasted from the spoil area.

The Sakuma Reservoir is a large reservoir for power generation. It has a total storage capacity of 330 X $10^6 m^3$ retained by a 155-m high dam on the Tenryu River of Japan.

A volume of 73 X $10^6 m^3$ of sediment accumulated in the Sakauma Reservoir in a period of 24 years between 1956, when it became operational, and 1980. This represents a 23 percent loss of storage capacity. The annual average rate of sediment deposition was 3 X $10^6 m^3$ p.a.

A total of about $10^6 m^3$ of sediment is removed annually to counter aggradation of the riverbed. The sediment dredged from the upstream section of the Sakuma Reservoir is of good gradation and is mostly of good-quality quartz sand highly suitable as fine aggregate for concrete.

Prevention of deposition in the back-up zone upstream from the reservoir is aided by maintaining the low-water level period for as long as possible.

The Miwa Reservoir was built in 1959, for the purpose of flood control, irrigation and power generation, on the Miwa Gawa Brook, a tributary of Tenryu River, Japan. The dam, 69.1 m high produced an initial total storage capacity of 37.0 X $10^6 m^3$. The original useful storage capacity was 25.5 X $10^6 m^3$. It was estimated in the design that a total of 6.5 X $10^6 m^3$ of sediment would be deposited in 40 years. This proved to be a considerable underestimate. In practice, between 1959 and 1972 sediment deposition amounted to 9.5 X $10^6 m^3$, which indicates that siltation proceeded at many times the predicted rate.

Dredging begain in 1965 and a total volume of 2.3 X $10^6 m^3$ had been excavated by 1974. The dredged material was used as aggregate for concrete. In October 1973, a pump dredger of capacity of 150 X $10^3 m^3$ p.a. was put into operation as part of a 10 year dredging plan to maintain the useful capacity of the reservoir.

4.4.3 Siphoning

Siphon dredging used for desilting reservoirs differs from ordinary suction dredging in exploiting the hydraulic head difference between the upstream and downstream levels of the dam as the source of motive power for the suction dredging. Three examples will be given:

(1) The simplest successful type of device is the example of hydraulic siphon device installed at the Rioumajou Dam in France (Evrard, 1980). The siphon straddles the gravity arch dam 21 m in height (Figure 4.63). The entrance of the siphon is located between the water intake and the sluice to clear sediment from the intake because the silting in the reservoir was such that every year solid sediment obstructed the bottom outlet and threatened the intake located 4 m higher. The siphon can operate automatically when the spillway functions. An automatic dewatering pot supplemented with a manual dewatering device is used for maneuvering. The upstream branch of 450 mm diameter is 20 m long and equipped with a priming nozzle near the outlet. The downstream branch of 400 mm diameter is 24 m in length. The siphon device can discharge 1 m^3/s with a carrying capacity of 15 kg of sediment. The siphon device of the Rioumajou Dam operates with remarkable efficiency and its cost was amortized almost within one year.

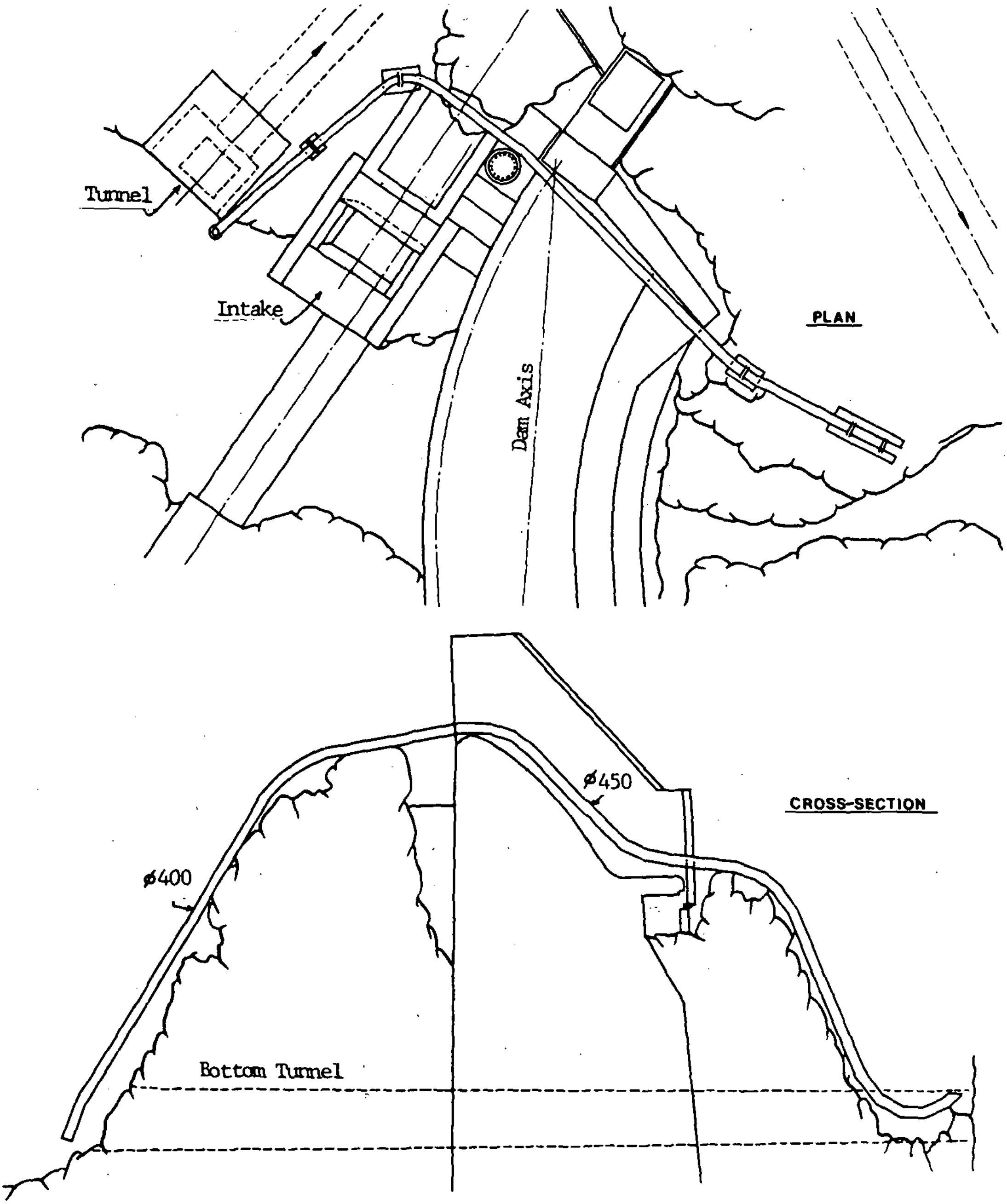


Figure 4.63. Hydraulic siphon device in Rioumajou Dam (After Evrard. 1980)

(2) Siphon dredgers have also been used in small reservoirs in north and northwest China for restoring storage capacity, e.g., the experimental siphon dredging used in the Tianjiawan Reservoir.

The Tianjiawan Reservoir was built in 1960 on the Fenghe River, a tributary of the Yellow River. It has a storage capacity of 9.42 X 10^6m^3. The annual runoff is 3.95 X 10^6m^3 and the annual sediment inflow is 250 X 10^3m^3. During the period from 1960 to 1973 a total of 4.0 X 10^6m^3 of sediment accumulated in the reservoir. Mechanical analysis of the deposits shows that within a distance of 500 m above the dam, the median grain diameter is 0.006 to 0.008 mm. Particles finer than 0.005 mm formed 40 to 48 percent of the material and particles 0.05 to 0.005 mm in diameter formed 44 to 58 percent of the sediment.

The siphon dredger has been in operation since 1975. During the period of June 1977 through June 1978, a total of 298 X 10^3m^3 of sediment entered the reservoir, while the siphon dredger removed 320 X 10^3m^3. The total working time in this period was 695 hours and the average output 460 m^3/hour, the slurry having a mean sediment concentration of 15.6 percent by volume.

The siphon dredger consists of three parts:

- An operation barge composed of 6 steel pontoons having an overall size of 8 m X 5.5 m X 1.1 m with a displacement of 16.8 tons. The power installation is 31 kW for driving winches.
- The 228.9 m long pipeline is made of steel pipes 550 mm in diameter, and contains soft tyre joints. The suction head of "Dustpan" shape with a scraper and nozzles, and later a rotating cutter is attached to the entrance of pipeline, supported by pontoons.
- The pipeline is connected to the bottom outlet of the dam by an underwater valve chamber (Figure 4.64). The total head of the Tianjiawan project is 17.7 m, but the effective head is only 7.9 m.

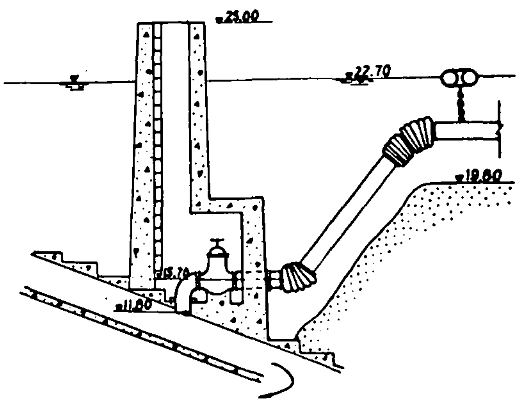


Figure 4.64. Sketch of outlet of dredging pipeline in Tianjiawan Reservoir

The advantages of the siphon dredging in Tianjiawan Reservoir can be outlined as follows:

(a) Low cost of dredging.

(b) There is almost no waste of water during the operation. The water sediment mixture discharged is used for irrigation and serves as fertilizer in warping areas.

(c) The siphon dredger is easily maneuvered, and has a high flexibility.

The shortcomings of such a siphon dredger are:

(a) Although the tyre joint used to meet the demand for a flexible joint is simple in construction and low in cost, it provides a high resistance to the flow. When the slurry has a volumetric concentration of between 1 percent and 11 percent, the coefficient of resistance of the tyre joint is 17 to 20 times greater than that of the steel pipe.

(b) Blocking in the pipeline may occur when the sediment concentration exceeds a certain limit, an abnormal distribution of pressure develops in the pipeline, or the slurry velocity falls below the critical velocity of deposition.

(3) "Jandin" method. The method for siphoning sediment deposits through the dam was first suggested by Jandin in the last century (Brown,
Engineer Jandin developed and used this method during the period 1892
1944). Engineer Jandin developed and used this method during the period 1892 through 1894, to remove sediment by siphoning material through the dam of the Djidiouia Reservoir in Algeria. In 3 years, $1.4 \times 10^6 m^3$ of silt and clay were removed; but of this amount, only 498,000 m^3 was thought to have been previously accumulated deposits, the remainder having entered during the years of operation. This reservoir, built in 1873 through 1875 for water supply, had an original storage capacity of $2.49 \times 10^6 m^3$, which was seriously depleted within 10 years of construction. The average content of sediment in the inflow was 3 percent, with a maximum concentration of 7 percent during flood flows.

Two sluice gates, 1.15 m in diameter, were installed in the base of the dam, but they were not adequate to prevent rapid silting, possibly due to incorrect operation.

Jandin's scheme consisted of a flexible pipe, 61 cm in diameter, capable of discharging $1.53 m^3.s^{-1}$ under normal operating conditions. It passed from an opening in the base of the dam to free-floating, sheet iron pontoons, which permitted the pipe to be moved around the reservoir within a radius of about 1.6 km. A turbine, installed near the mouth of the pipe and actuated by the pipeline flow, was coupled to a wheeled chopping instrument near the intake end of the pipe, which was designed to stir up the sediment.

A new method developed by Hannoyer (1974) is based on the principle of "hydroaspirator", proposed by Jandin 80 years ago.

Figure 4.65 is a schematic diagram of the arrangement of the hydroaspirator. A flexible pipeline is connecting the bottom sluice. The head part of the pipe is movable so that the sediment deposits may be siphoned out by the water head above the outlet level. No pump is needed.

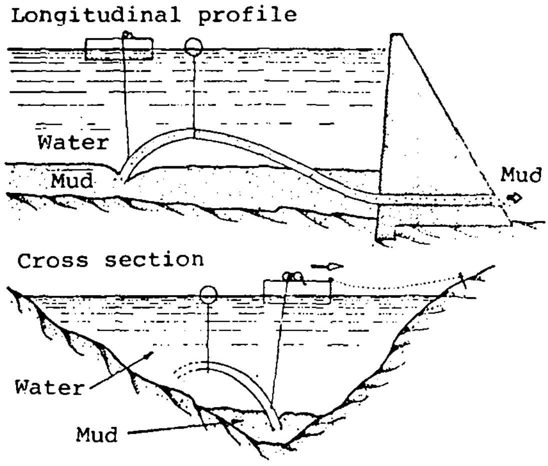


Figure 4.65. Schematic diagram of the arrangement of siphon dredging (After Hannoyer, 1974)

The length of the pipeline which is suspended by float to keep it located above the mud surface may be more than 2 km.

Experiments using siphon dredging were also carried out in the Bongival Reservoir on the Seine in 1970.

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Chapter 5
Prediction Methods

5.1 OBJECTIVES AND SCOPE OF PREDICTION

5.1.1 The Objectives

Prediction of reservoir sedimentation is always objective oriented and serves to satisfy definite demands of planning, design or operation of water resources systems. The objectives of prediction will be briefly examined in this paragraph, since the methods of prediction depend greatly upon these objectives.

The objectives are different in the various stages of the water resources project, as shown in Table 5.1.

In the planning stage the main objective is to estimate the length of the expected useful life of the reservoir, which is most important in the economic justification of the planned water resources project. The prediction requires an estimate of the average amount of sediment which would enter the reservoir and the part of it which would be trapped by the reservoir. In certain cases, the emplacement of the sediment deposits within the reservoir may affect the allocation of the storage capacity to the different users. Hence, a preliminary estimate of the development of the sediment deposits and their distribution in the reservoir may also be required at an early stage of the planning.

The cost estimate of the planned project may contain conservation measures to be implemented in the watershed of the reservoir, in order to reduce the sediment yield. The extent of such works would depend on their positive effects on reducing the sediment yield and reservoir sedimentation. Hence, the preliminary estimates of the expected sedimentation would contribute to the planning of the necessary conservation measures and works.

In certain cases the detrimental effects of sedimentation upstream and downstream of the dam can be so great that the project is economically not wise. These impacts should be estimated and included in the cost of the project.

The intensity of sedimentation depends to a certain extent on how the storage is used. Therefore, a preliminary outline of the planned operation of the reservoir may be a necessary part of sedimentation studies in the planning stage of the project.

In the design stage of the project, the objectives of the prediction of reservoir sedimentation are manifold. In the first place, the dam and appurtenant structures must be designed having in view the effects of sedimentation in the reservoir as well as the needs of sediment control (flushing of sediment deposits, venting of density currents, by-passing of sediment-laden flood waters, etc.). These considerations concern the position and size of the intakes, spillways, bottom outlets, sluices, gates, etc.

Since sediment control works in the watershed and particularly those protecting the reservoir from bank erosion, gullies, torrents, etc., may represent an important item of the cost estimate of the project, these works must also be designed together with the dam and other major structures of the project. The

Table 5.1 Objectives of Predicting Reservoir Sedimentation

A. In the planning stage

Prediction of the life of the reservoir.

Estimate of the effects of reservoir sedimentation on storage allocation.

Planning for and estimates of the effectiveness of conservation measures in the watershed.

Estimate of the magnitude of secondary effects upstream from the dam and provision for adequate measures.

Estimate of the magnitude of the effects downstream of the dam and provision for adequate measures.

B. In the design stage

Design of the dam and appurtenant structures from the point of view of sedimentation.

Design of sediment control in the catchment.

Calculation of secondary effects upstream of the dam and design of adequate works or measures.

Calculation of the effects downstream of the dam and design of adequate works of measures.

C. In the operational stage

Development of operational techniques for flushing, venting of density currents, etc., and conduct of such operations.

Evaluation of remedial measures and reconstruction of the structures according to need.

Real time forecasting of sedimentation processes for improved planning of the future use of the reservoir.

design of works to reduce reservoir sedimentation depends greatly on the reservoir sedimentation studies.

The operational rules of the reservoir which affect the sedimentation in the reservoir and the application of sediment control techniques are worked out in view of the needs of the various users of the storage (e.g., for water supply, hydropower production, irrigation, flood control, etc.). In many cases, the analysis of the operational rules from the point of view of reservoir sedimentation is an important part of the design.

Sedimentation in the upper parts of the reservoir will cause additional increase of the water levels, which increases in time, as the deposits progress. In some cases, this may have an important effect on flood control works (levees and dikes, bridges, port structures) and may cause additional expenses to be attributed to the project in the future. The design of the dam and reservoir must take into account these effects and foresee technical and other solutions, which in turn, depend greatly on the prediction of reservoir sedimentation. Hence, the analysis of secondary effects of reservoir sedimentation (the primary being the loss of storage) is very often an important task of the design. The same holds for the effects downstream from the dam (scour and deepening of the river bed, with effects on bank protection works, intakes, river ports, etc.), which are also the subject of reservoir sedimentation studies in the design stage.

In the operational stage, sedimentation studies are conducted for various objectives. In the first place, the predictions made in the design phase should be checked by comparing them with measurements and observations after completion of the dam. Instead of simulation, real time forecasting of the sedimentation may be developed as required by sedimentation control measures.

The usual control measures consist of flushing the deposited sediment, venting of density currents, etc. The techniques of these operations must be worked out on the ground of sedimentation studies and checked by the actual effects obtained in practice.

If necessary, remedial measures may also be envisaged, consisting of the reconstruction of parts of the dam and appurtenances, such as the enlargement or the increase of the number of bottom outlets and sluices, provision for by-passing the floods, implementation of additional sediment control works in the catchment, construction of dams upstream or sills downstream from the reservoir, etc. All these measures call for a detailed study of the actual sedimentation processes in the reservoir.

The methods to be applied in the sedimentation studies depend very strongly on the objectives.

In the planning stage, an assessment of the sediment yield in the catchment and an empirical estimate of the trap efficiency of the reservoir may suffice, sometimes combined with a rough estimate of the position of the expected deposits within the reservoir.

In the design stage, more detailed calculations are required; mathematical modeling of the reservoir and a calculation of the sediment balance along it is typical for such studies. Comprehensive field investigations are required for these studies, to supply the input data to the model and if possible, to calibrate the formulae and parameters of the calculation.

In the operational stage, field measurements and observation, as well as their comparison with the predictions, are essential. The models used must be validated, in order to develop them as tools for the real time prediction needed for sediment control operations (flushing, venting of density current, etc.).

5.1.2 Decisions Based on Reservoir Sedimentation Prediction

In general terms, the objective of decisions on reservoir sedimentation is to minimize the expected risk of inadequate performance of the system, through optimal choice of decisions by which the system performance can be influenced. Prediction of reservoir sedimentation is thus needed in order to be able to make the right decisions to mitigate the consequences of sedimentation on the performance of the water resources system, of which the reservoir is an essential part.

The physical system considered here is the river (or rivers), which provides the inflow of water and sediment into the reservoir, the reservoir itself, which transforms the inflow into the outflow of water and sediment and retains a part of the sediment, and the downstream part of the river, which is subject to morphological changes caused by the modified flow conditions of the river.

The prediction of the performance of this system is based on the performance of a model, which can be very simple, as it is in the case of empirical methods of prediction, but also very complex, requiring highly sophisticated methods of mathematical or scale modeling. The complexity of the model depends upon the requirements which have to be met in order to reach the right decisions with regards to the system performance.

The state of the physical system is determined by observations and measurements of the water and sediment inflow and of the river and reservoir characteristics. After the reservoir has been put into operation, the performance of the system can also be determined by observations and measurements of the outflow and of the changes in the reservoir characteristics.

The information obtained by measurements is limited because of technical, financial and other constraints; except for the recording of water levels, and of surveying the reservoir topography, all other quantities are measured or observed by sampling. Moreover, the measurement methods contain errors caused by different factors, some of them inherent to the methods of measurement, others random. The incomplete information and errors are reflected in an imperfect knowledge of the state of the system.

The model of the system consists of sets of theoretical or empirical relationships by which the reservoir performance can be simulated. The computational tools range from empirical diagrams to computer program packages and in some cases include physical scale models. The relations contain numerous assumptions, hypotheses, simplifications and empirical or semi-empirical coefficients; and, thus, represent the performance of the physical system in an imperfect way.

Prediction is thus hampered both by imperfect knowledge of the state of the system and by errors generated by the model itself. The value of the model is determined by its capability to predict the system performance with sufficient accuracy to properly make the required decisions.

The interaction between prediction and decisions is illustrated by Figures 5.1 (for planning and design stage) and 5.2 (for operational stage).

In the planning and design stage, decisions are made by the planner or designer concerning the site and the capacity of the reservoir, its design features, preliminary operational rules, etc., based on the predicted output. Since the reservoir does not yet exist, verification of the predicted performance is not possible. The many uncertainties of the input and model should be assessed by a sensitivity analysis in which all the identified and relevant parameters are varied within appropriate limits and the effects of these variations on the prediction checked. The analysis should also include the planning and design decisions which are influenced by the predictions. It is important to underline this, because the prediction of sedimentation is much more sensitive to the input and model uncertainties than the decisions themselves. Because of the slowness of sedimentation, in many cases the planning or design features of the reservoir will vary little despite considerable differences in the predicted sedimentation. It may take many years before the effects of sedimentation are felt and the economic impacts of future events influence present decisions in a reduced manner. As a consequence, very crude models may satisfy the requirements of decisions in the planning stage. Often good design decisions may be based on prediction by comparatively inexact modeling.

If the sensitivity analysis shows that design variables are substantially influenced by the uncertainties of the modeling, greater care in the predictions is called for. Improvements in the predictions may require improvements in both the input simulation and the model itself. As indicated on Figure 5.1, instructions will be given in order to: a) improve the observations and measurements on which the input simulation is based and b) improve the calculation methods used in the model. The process has to be repeated, at least in principle, until satisfactory confidence in the decisions for planning and design is achieved. In practice, financial and time constraints usually oblige the planners and designers to make final decisions without having fully explored the problem of reservoir sedimentation. In such cases, they should at least be aware of the possible consequences of their decisions.

Once the dam has been built and the reservoir put into operation, the actual output of the system can be assessed by direct observations and measurements for comparison with the predictions. Thus, the overall model can be calibrated and its parts verified by specific measurements. The uncertainties of input simulation can be partly eliminated by imposing the actual sequences of water and sediment inflow on the model, as observed after the reservoir has been put into operation; this allows the effects of errors in input simulation and model performance to be separated.

The decisions in the operational stage of the reservoir concern the operational rules and management practices. By appropriate management, the amount of sedimentation can be reduced and the deposits can be moved towards or into the dead storage. Such practices, however, may contradict the demands of the users of the stored water so that a compromise has to be reached. A comparatively accurate prediction of sedimentation is thus required in these studies. Such accuracy can only be achieved by combining field investigations with improved mathematical modeling, and very often, combining them with scale models.

In some cases, expensive remedial measures have to be applied, as explained in Chapter 4. Decisions on such costly measures should be taken only on the ground of reliable prediction of sedimentation in the actual and planned future status of the reservoir. Such predictions are possible only by properly calibrated models.

The interaction of prediction and decision making in the operational stage of the reservoir is indicated on Figure 5.2. In the process of calibration, instructions to improve observations, field measurements and model techniques can be

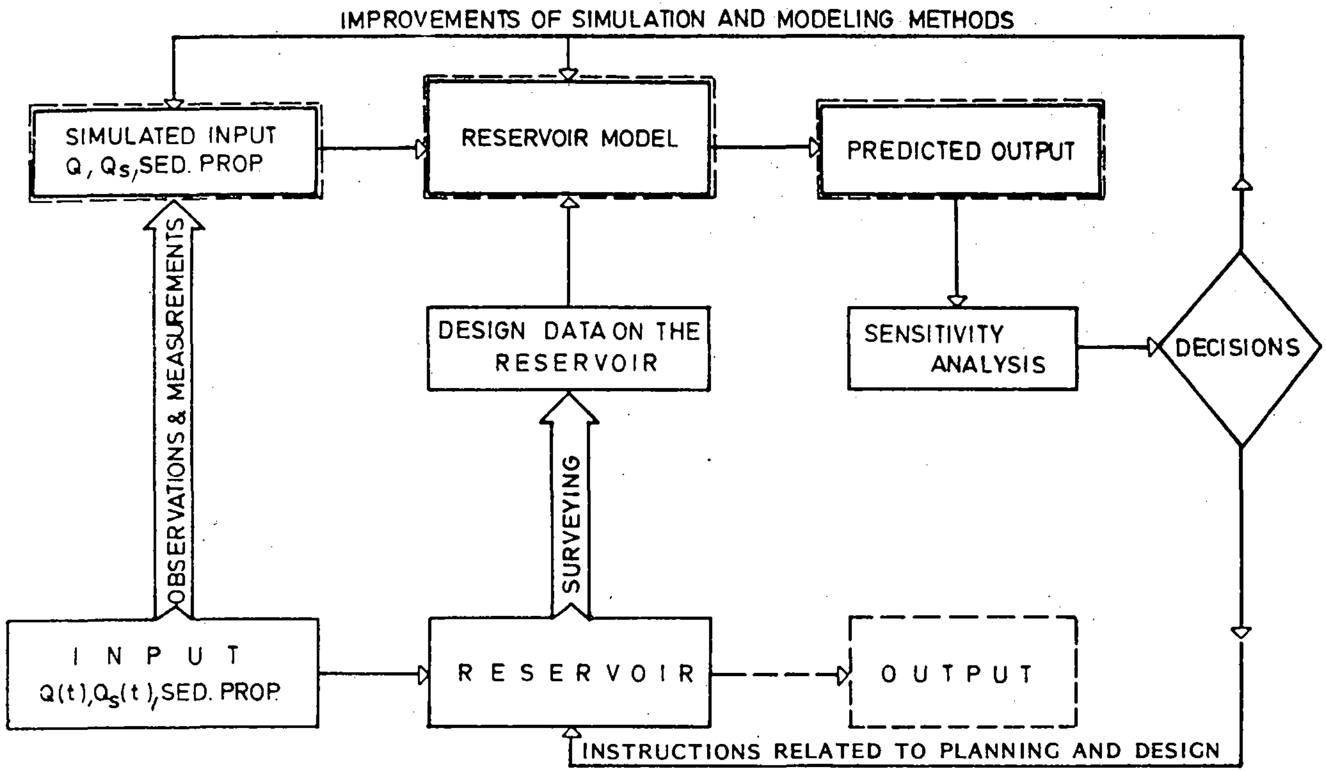


Figure 5.1. Prediction of reservoir sedimentation in planning and design stage

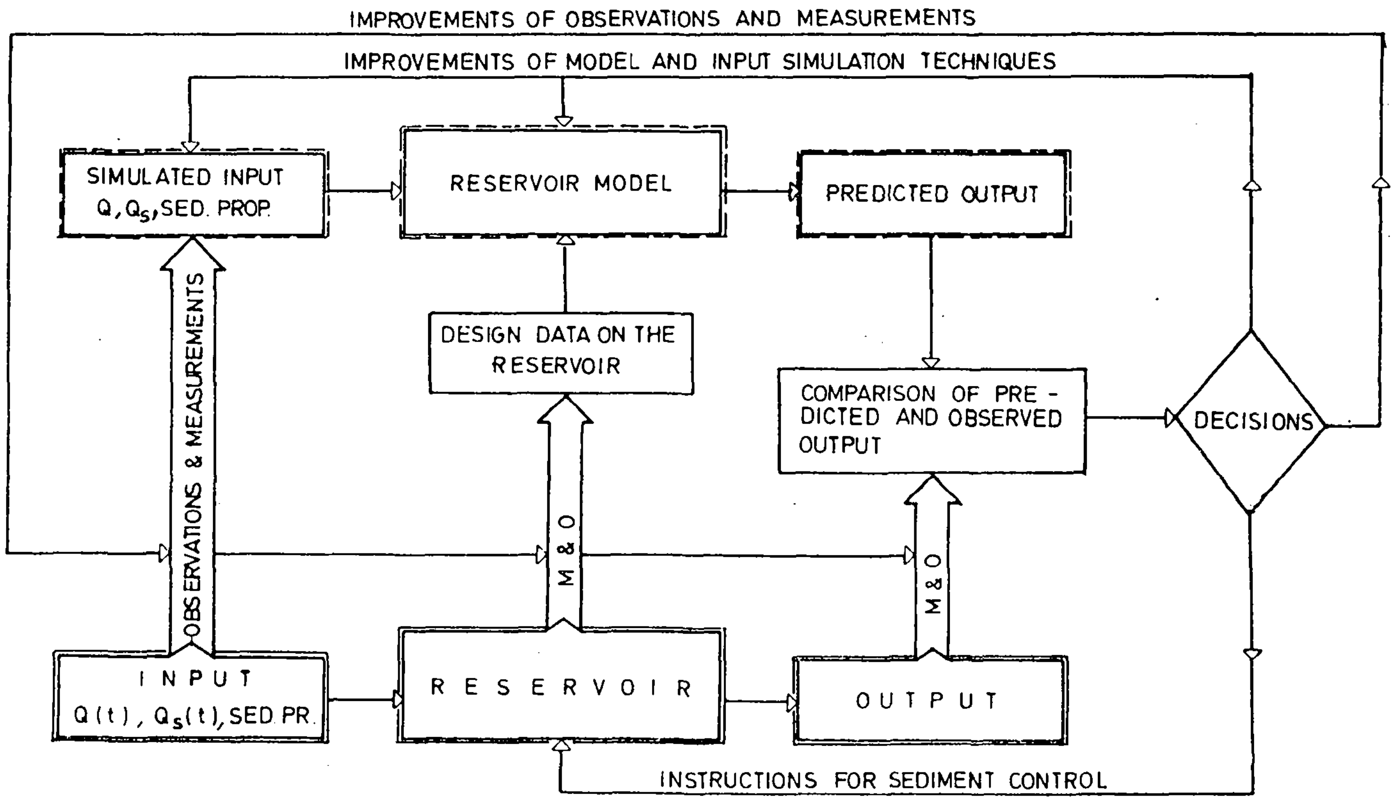


Figure 5.2. Real time prediction of reservoir sedimentation

issued after having compared the predicted and observed performance of the system. When the accuracy of the model has been proved, it will give predictions on the basis of which decisions can be made to improve the reservoir performance.

5.1.3 Probabilistic Character of Prediction

Predictions of reservoir sedimentation are inherently probabilistic. This means that no firm prediction can be made about the future state of the reservoir, but only a statement of probability of it. This should never be overlooked and it should be underlined that quasideterministic methods of predictions are usually applied in planning and design only because of the limited time and resources allocated to sedimentation studies. A full statement of probability would obviously require a more extensive analysis than a simple prediction based on averages and assumptions.

There are three main causes for the probabilistic character of the predictions:

1. *The stochastic character of the hydrological input*

The discharge of water, the sediment content and the grain size distribution of sediments vary in a stochastic way with time. In sedimentation studies they have to be simulated on the basis of past records, measurements, and observations. The replacement of the stochastic input by a deterministic one (e.g., by repeating past series of inflows) is an expedient justified only by the limitations imposed on the studies.

2. *Stochastic linkage between the flow of water and sediment transport*

The flow of water through the reservoir is described by hydrodynamic equations which ensure a deterministic link between inflow and outflow. To the contrary, the relationships describing sediment transport as a function of the flow of water are by no means deterministic: the initiation of motion, the development of bed forms, the transport of bed-load or suspended sediment and the settling of sediment particles - are all stochastic phenomena. The deterministic relationships which are used in the calculations are approximations or at best, only express the relationships between mean values of the parameters.

3. *Imperfect knowledge of the state of the system*

As mentioned in paragraph 5.1.2, the measurements and observations by which the input and the state of the river and reservoir are defined give an imperfect image of the system.

Observations and measurements of the flow of water and sediment are made by sampling at selected points and time and even a continuous record of water levels must be digitized before using it in the model. Similarly, a survey of the river and reservoir topography is almost always based on discrete measurements (e.g., cross sections), which are then built into the model. The composition of the sediment in the river bed and in the reservoir is also defined by discrete sampling. Any sampling is localized in space and time and however precise and comprehensive the sampling may be, there will be differences between any two sets of samples. In view of the limited resources and time available, the sampling of some variables (particularly connected with sediment transport) may be very crude and thus introduce a considerable amount of error in the information obtained. Quite often, analysis would show that more detailed sampling is needed to improve the prediction.

There are certain important variables of the sedimentation process which can be measured in the field only very imperfectly, e.g., the transport of bed load, the density of unconsolidated deposits in deep water, etc. The information obtained on these variables is scarce, uncertain and unreliable in most cases. The values used in the model must very often be assumed, taken from flume experiments or from empirical information from other reservoirs. A considerable error is thus introduced into any model and a sensitivity analysis is the only means of assessing the effect of the error on the outcome of the calculations. The sensitivity analysis will bring to light the most important predictors of the process and by additional effort, the initial error can be reduced.

5.2 . PREDICTION OF RESERVOIR SEDIMENTATION

5.2.1 Empirical Methods

Empirical methods of prediction are based on observations and field measurements made on many reservoirs around the world. Their scope is limited to a few features only, which are important in the preliminary evaluation of reservoirs in the planning and feasibility study stages. These are:

a) an estimate of the total amount of sediment which will be retained in the reservoir in a certain period of time, and

b) a rough estimate of the location of the bulk of the deposits in the reservoir.

The ratio of the sediment retained by the reservoir to the total inflow of river sediments is usually called the "trap efficiency" of the reservoir. Based on empirical data, the trap efficiency is correlated to various parameters, such as:

- the ratio between the reservoir capacity and the annual inflow of water into the reservoir
- the retention period, defined by dividing the reservoir volume by the daily inflow rate
- the specific storage of the reservoir, defined as the ratio between the reservoir volume and the river basin area controlled by the reservoir.

Additional refinements may also be introduced, such as classifying sediments as coarse or fine, making a distinction between the local production of sediment and the silt which comes from upstream reservoirs. A very complete review of such methods is given in the book of Bogardi, 1971.

Two methods of American origin are widely used to estimate the trap efficiency: the method of Brune, 1953, which is based on the capacity/inflow ratio and the method of Churchill, 1948, which is based on a sedimentation index obtained by dividing the retention period by the average flow velocity in the reservoir. More than 30 years have passed since the first appearance of these studies but the experience with reservoir sedimentation since that time has not been systematically compared with the findings of these authors. The same diagrams are repeated in many more recent textbooks or manuals but apparently no attempt has been made to complete or modify the diagrams on the basis of new data.

Both of the above methods are described in easily accessible reference books, such as the ASCE Manual "Sedimentation Engineering" 1975; the textbook of W.H. Graf, 1971; the book of Simons and Senturk, 1978; the book edited by H.W. Shen, 1971; and the publication of B.N. Murthy, 1977. A full account of these methods will therefore be avoided here. Figure 5.3 reproduces the well-known curve of Brune. It can be seen that for a capacity/inflow ratio of greater than 0.1 the trap efficiency is over 70%, and for a capacity/inflow ratio of 0.2, it reaches 95 to 100 percent. These relations have been confirmed in many cases and they show that very large reservoirs retain virtually all the incoming sediment. This consideration is important in the planning stage of large reservoirs and the more precise calculations of the distribution of deposits within the reservoir can be left for later stages of the design. For small capacity/inflow ratios, the method of Brune is less reliable, which is logical since the hydraulic conditions in small reservoirs vary greatly, depending on the topography, hydrology and sediment properties.

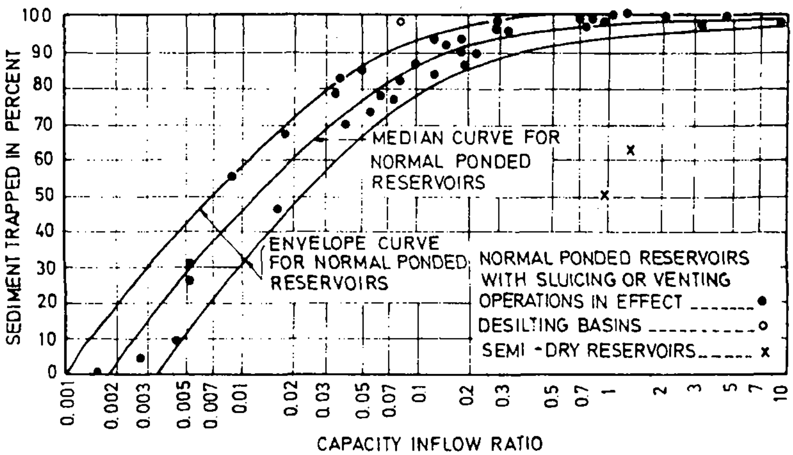


Figure 5.3 Trap efficiency, after Brune, 1953

The sediment retained by the reservoir will be partly deposited within the "dead storage" which is not used in operation, but partly in the operative storage as well. Empirical methods have been proposed to estimate the spatial distribution of the deposits by diagrams. The best known of these were proposed by Borland and Miller, 1958, and have been supported by Bondurant, et al., 1978. The diagram in Figure 5.4 shows the ratio of silt deposited in the dead storage to the total amount of silt retained by the reservoir, as a function of the ratio of the dead storage to the total capacity of the reservoir. The authors divided the examined reservoirs in four groups:

Type I - Lake
Type II - Flood plain in foothill
Type III - Hill
Type IV - Gorge

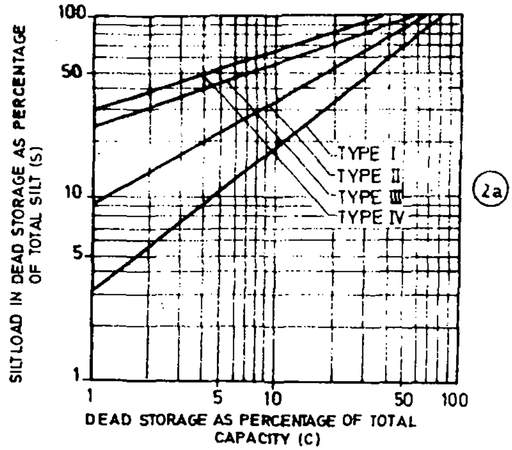


Figure 5.4 Silt deposit in dead storage as percentage of total silt deposited (after Borland and Miller, 1950)

The diagrams are based on surveys of 51 reservoirs in the U.S.A. It can be seen from them, for instance, that when the dead storage is about 10 percent of the total capacity, the sediment retained in the dead storage will range from 15 to 70 percent of the total sediment deposited, depending upon the type of the reservoir.

Other information on the spatial distribution of deposits has been given by Murthy, 1977, as shown in Figure 5.5. This figure is based on the data of four reservoirs in India. It can be seen that except for the very deep reservoir of Bhakra, deposition occurs mostly in the upper parts of the reservoir: about half of the sediment is deposited in the shallow part of the reservoirs, where the depth ranges from 20 to 30 percent of the maximum depth. B.N. Murthy, 1977, gives valuable instructions on how to use the empirical methods in practice.

From the user's point of view, the most important deposits are those in the live storage, at the upstream end of the reservoir. Hydraulic conditions in that part of the reservoir, however, vary through very broad limits, depending on topography, hydrology, sediment properties, etc. Prediction by simple empirical diagrams can hardly be effective in such cases, except for very preliminary estimates.

The general pattern of reservoir sedimentation is described by the exponential law, as proposed by Lapshenkov, 1979:

$$V_{x,t} = V_{x,\infty} \left(1 - e^{-\frac{t}{E_{x,t}}}\right) \tag{5.2.1}$$

where:

$V_{x,t}$ - volume of deposit at reach x and time t

$V_{x,\infty}$ - ultimate volume of deposits at reach x

$E_{x,t}$ - a sedimentation index, having the dimension of time. It depends on channel geometry, flow hydrology and sediment properties.

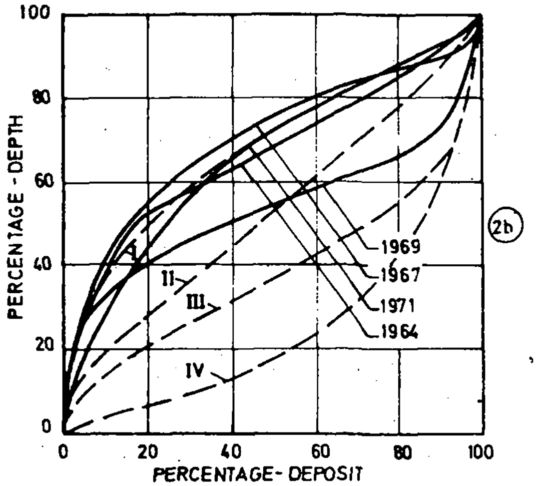


Figure 5.5 Relative distribution of deposits in reservoirs, after Murthy, 1977.
I,II,III and IV - Type curves after Borland and Miller, 1950.
1964 - Panchet Hill; 1967 - Nizamsagar; 1969 - Bhakra; 1971 - Maithon.

This exponential law describes qualitatively the progression of sedimentation in time. The ultimate volume of the deposits is the value which would be attained after a long period of operation of the reservoir, that is after a new equilibrium has been established. This volume can easily be estimated by subtracting the volume of the former river channel from the initial reservoir capacity.

Lapshenkov gives indications of how the index E can be determined from known values of the parameters. This determination involves a combination of analytical and empirical methods. The value of the index could be, in principle, determined empirically from the equation 5.2.1 itself, if the volume of deposits $V_{x,t}$ is known after a time t. Once the index $E_{x,t}$ (which may vary from reach to reach and also with time) is known, prediction of future sedimentation in the reservoir is possible by the same equation. This may be useful in the case of reservoirs on mountain rivers with flash floods, transporting large quantities of sediment in very short periods of time. Neither the floods nor the sediment transport can be measured on such rivers, but surveying of the reservoir bed is feasible (e.g., by aerial photography) since the reservoir is empty for the greater part of the year. On the basis of the successive measurements of the reservoir topography, the sedimentation index $E_{x,t}$ could be determined and the equation used for estimates of future sedimentation.

5.2.2 Mathematical Modeling of Reservoir Sedimentation

The mathematical modelling of reservoir sedimentation is based on the equations of motion and continuity for water and sediments over a mobile bed.

In the past years, attempts have been made by several authors to develop models of various degrees of sophistication.

A complete model of reservoir sedimentation would require the mathematical description of the three-dimensional field of flow of water and sediment transport. This is still beyond the possibilities of computational hydraulics. Physical (scale) models have to be built for cases where it is believed that a three-dimensional model is necessary. The modeling is then based on the principles of mobile bed similarity.

Recent techniques of mathematical modelling of turbulent flow (Rodi, 1981) make the solution of two- and three-dimensional fields of flow possible, within certain limitations. In principle, it is possible to attach a sediment balance calculation to such solutions and by this to enlarge the scope of mathematical modelling of reservoir sedimentation. Attempts by Hauguel, 1977, and Ariathurai and Krone, 1976, though not intended for reservoir sedimentation, open the path for such applications.

For most practical purposes, however, one-dimensional models provide a satisfactory answer, and most of the work on reservoir sedimentation modelling relates to such cases. The models described by Yücel and Graf, 1973; Thomas and Prasuhn, 1977; Cunge and Perdreau, 1973; Bruk and Miloradov, 1968, 1977, 1980; and others, mentioned in literature (by Karaushev, 1961; Bogardi, 1971; Simons and Sentürk, 1977; etc.) seem to agree with regards to the simplifications and assumptions which make their use feasible. These will be described below.

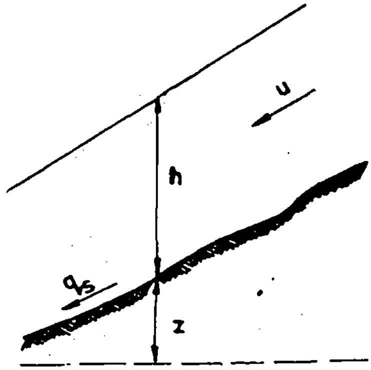
The main simplifications and assumptions commonly used in models are based on the following considerations:

a) Predominance of the length of the reservoir over the other two dimensions. Since the length along the impounded reach of the river usually exceeds the width and the depth of the reservoir, the one-dimensional approximation of the process is justified. The equations for the flow of water and sediment are then used in a form integrated over the cross section. (Figure 5.6).

b) Difference in the rate of change of water levels and bed levels. The variation of water levels with changing discharge is several times faster than the response of the river bed to these changes (de Vries, 1973 and 1977). This makes it possible to solve the equations for the flow of water and of sediment movement separately, in sequence instead of simultaneously.

c) Smallness of the inertial terms in the equations for the flow of water. The one-dimensional equations for the flow of water are given in the form of the well-known Saint-Venant's equations. Since the variation of discharge in rivers with time is rather slow, the inertial term of the Saint-Venant's equations is much smaller than the other terms. As a consequence, water levels can be calculated by a sequence of steady state equations.

d) Imperfect knowledge of sediment transport. The laws governing the movement of sediments are not known exactly, or at least, much less exactly than those for the flow of water. Hence, empirical expressions have to be used linking the transport of sediments with the flow of water. These relationships are mainly obtained from laboratory flume experiments in uniform flow conditions and they contain a large margin of error. Therefore, refinement of the



THE SAINT VENANT'S EQUATIONS:
(FOR THE FLOW OF WATER)

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + g\frac{\partial h}{\partial x} + g\frac{\partial z}{\partial x} = -g\frac{u|u|}{c^2 h}$$

$$\frac{\partial h}{\partial t} + u\frac{\partial z}{\partial x} + h\frac{\partial u}{\partial x} = 0$$

THE EQUATIONS FOR SEDIMENT TRANSPORT:

$$q_s = f(u, c, \rho, \rho_s, d, etc....)$$

$$\frac{\partial z}{\partial t} + \frac{\partial q_s}{\partial x} = 0$$

Figure 5.6 The equations for unsteady flow of water and sediment

sediment transport equations for nonuniform or unsteady flow does not seem to be justified.

e) Very irregular reservoir topography. Only numerical methods can be applied, proceeding in step-by-step calculations, from reach to reach.

The general pattern of sedimentation modeling in reservoirs can thus be described as follows (Figure 5.7):

1) For a constant discharge (the first in a given series of constant discharges representing the inflow hydrograph) the backwater profile is calculated by a standard method, starting from the downstream end of the reservoir, where the boundary conditions are defined by a known head-discharge relationship.

2) Sediment transport is calculated, reach by reach, beginning at the upstream end and marching downwards, making use of the available sediment transport formulae (for bed-load, suspended load or total load). At the upstream end, the inflow of sediment should be known from hydrology.

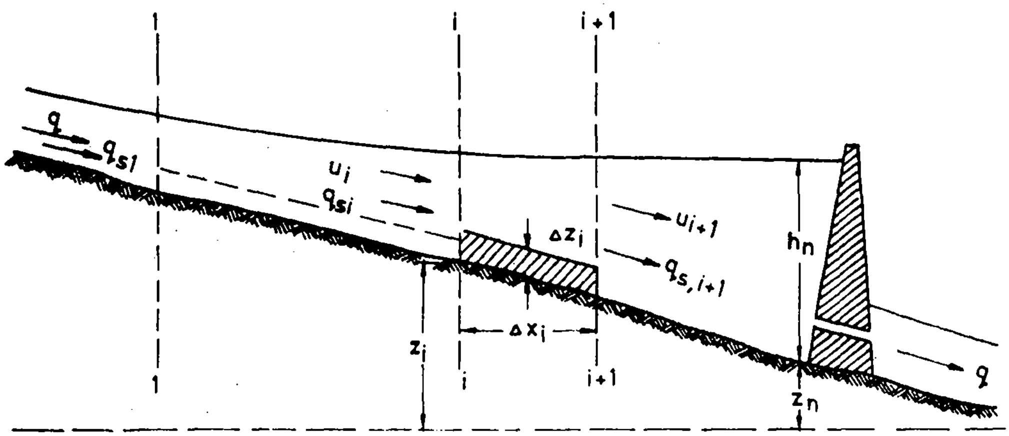


Figure 5.7 *Definition sketch for sedimentation calculation*

3) From reach to reach, the inflow of sediments is balanced, assuming that the difference between inflow and outflow will be deposited within the reach (or in the case of a negative balance, eroded from the reach).

4) The balance of sediments (expressed in weight or volume, depending on the formula used) is converted into a modification of the reservoir bed (aggradation or degradation).

5) The calculation is then repeated with the new geometry of the reservoir and the subsequent input discharges of water and sediment.

Many models have been worked out according to the above scheme. The flow chart of one of these, developed at the Hydrological Engineering Center in Davis, U.S.A., is shown in Figure 5.8. This model, which is known under the name of HEC-6, was developed by Thomas and Prasuhn (1977), and can be obtained from the U.S. Corps of Engineers.

In spite of its apparent simplicity, the application of such a model is always a difficult problem, which should be approached with much caution and care. It is important to remember that because of the limitations of the basic equations and the many assumptions and simplifications, no model can be of general validity.

A model, developed or applied for a specific case, should always be objective oriented, i.e., focused on certain problems, relevant to the engineering objective of the investigations. The model should be based on a sound understanding of the phenomena, avoiding unnecessary detours and complications, which could contribute but little to the solution of the main problem. The data on which the model is based should be obtained by objective oriented field investigations rather than from textbooks.

Without entering into details, which are discussed in the references quoted, the requirements and limitations of the one-dimensional reservoir sedimentation models will be described in the following points.

Field Data

The data on which the model is based are obtained by field investigations, and they consist of the following:

Survey of the reservoir topography, at a given time which is then taken as the starting point of the calculations. In the planning and design stage, topographic maps of appropriate scale can be used, except for the river bed itself, which usually has to be surveyed. Aerial photographs are very useful because they give information on vegetation and the state of the surface as well.

Water level observations and measurements at several points along the rivers feeding the reservoir, the reservoir itself and downstream from it. Observations which are regularly made at gauging stations usually have to be completed with additional points, needed for the calibration of the model.

Water discharge measurements, related to the water level observations (at gauging stations or additionally to them).

Sediment investigations. Bed load measurements are at the prsent state-of-the-art unreliable, so that suspended load discharge is about all that can be determined by field measurements. The discharge of bed load can be estimated on the basis of the grain size distribution of the bed-material, by applying appropriate sediment transport formulae. The determination of the grain size distribution of suspended load is essential in any sediment study. Measurement of bed forms, such as dunes, shoals, etc., is important. Another of the important parameters of sedimentation calculations is the determination of the density of the bed materials, and in existing reservoirs, of the sediment deposits.

Derived Values

The basic field data have to be processed in order to make them applicable to the one-dimensional model.

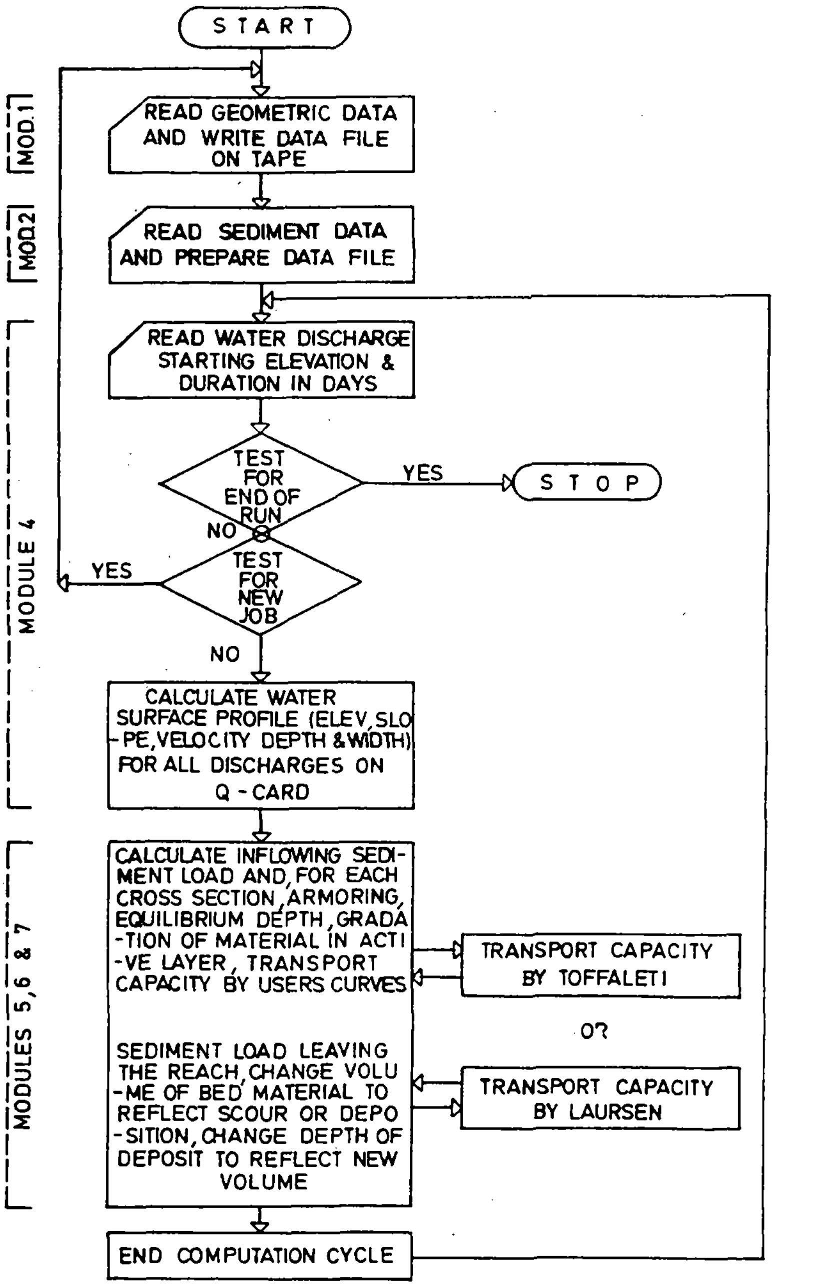


Figure 5.8. Flow chart of the HEC-6 Model (Thomas and Prasuhn, 1977)

Values characterizing the channel geometry

The composite shape of the river (reservoir) channel has to be expressed by the values used in one-dimensional hydraulics: the cross sectional area A, the channel width B, the mean depth H. The wetted perimeter can usually be equated with the channel width and the hydraulic radius with the mean depth.

The shape of the natural river channel is irregular and for the purpose of calculation, the cross section has to be divided into parts, separating the main channel from the floodways, secondary channels, and branches, if necessary. The zones without flow (dead zones) should also be separated. In meandering rivers, the lengths of the river reaches can be different for the main channel and the floodplains, varying also with the stage.

The geometrical representation of the river channel is a very important task, which influences the result of the calculations. In certain cases, when the consequences are of great engineering importance, the use of scale models may be justified.

Values characterizing the flow of water

From the direct measurements of the flow velocities, the discharge is calculated and when the channel is divided into parts, given separately for each part. The mean velocity is determined from the discharge and it is naturally different in each part of the composite channel.

According to the needs of the hydraulic calculations, further values characterizing the flow are defined, such as the shear stress, shear velocity, etc.

The channel resistance or roughness is one of the most important predictors of the one-dimensional flow calculations and should as a rule be determined from field measurements by calibration. It is expressed by a roughness coefficient (as in the Chezy, Manning, Darcy-Weissbach or other uniform flow formulae), the value of which varies from reach to reach, changes with stage and is different in the different parts of the channel.

Since the values of the roughness coefficient depend on the way the river channel geometry was schematized, the same type of schematization should be used in the predictions as was used in the calibration.

Values characterizing the movement of sediments

From the measured suspended sediment concentrations and flow velocities, the discharge of suspended sediments is calculated for the different parts of the channel and in total. In the view of the hydraulic calculations to be performed, the discharge should be expressed by grain size fractions, and the weighted average grain size of the suspended load determined. For suspended sediments, the most useful way of defining the grain size is by the settling velocity of the particles, which is directly measured on the samples taken. The corresponding values of grain diameters are then obtained, if necessary, from charts, tables or formulae.

From the samples of bed material and their grain size distribution, the average grain size distribution of bed material along the river has to be determined (for the parts of the channel as used in the calculations).

The grain size distribution may further be characterized by quantities, such as D_{10}, D_{50}, D_{90}, etc. - meaning the diameters of which the indicated percentage of material is smaller (or greater).

WATER SUPPLY AND

Auxiliary functions and correlations

Since the measurements planned and executed for a specific engineering design cover usually only a short period of time (a few years), there is a need to extrapolate the data to a longer period. This may be achieved by correlations between the relevant parameters, so that relationships are obtained which may then be used in the model.

Typical functional relationships or correlations used in order to fill the gaps, extrapolate or complete the input data are:

- relationships which express the variation of the channel resistance (roughness) as a function of the variations of stage or discharge.
- relationships expressing the variation of the suspended load discharge as a function of the discharge of water.
- dependence of the grain size of suspended load (expressed through the weighted average settling velocity or grain diameter of the suspended sediment) upon the discharge of suspended sediment.
- relationships which indicate the influence of the discharge on the average diameter of the bed material.
- dependence of the density of deposited sediment on grain size, water depth and age of the deposits.

Since all such relationships are founded on the correlation of empirical data, there is a great scatter of points; and refinements are sometimes introduced. A statistical analysis of the correlations can be recommended in order to define the confidence bands, which may later be used in the sensitivity analysis of the model performance.

Input simulation

The input into the model consists of series of discharges, of water and sediment at the entry section (or sections, if the reservoir is fed by more than one river). The sediment discharge should be given separately for suspended and bed load, and the grain size of each has to be specified as needed for the subsequent calculations.

Historic records of water and sediment discharges are usually too short or incomplete, so that the series have to be produced by making use of the correlations mentioned above. Instead of historical, simulated series may also be produced following certain criteria of linkage between the statistical properties of the historical series of the various magnitudes. The confidence in simulated series is, however, rather low, because of the many variables linked together with loose correlations.

Boundary conditions

Since flow in reservoirs is subcritical, only the downstream boundary condition need be known for the backwater calculations. It is given by a head-discharge relationship, governed by the dam at the downstream end of the reservoir and reflecting the operational rules of the reservoir.

Sediment Transport Formulae

The prediction of reservoir sedimentation depends to a great extent on the sediment transport formulae used for the sediment balance calculations. These contain, however, a large margin of uncertainty.

Bed Load Formulae

The main difficulty of bed load formulae is that in rivers bed load measurements are highly unreliable, if not impossible (like during important floods). The formulae were, therefore, developed on the basis of laboratory flume data, and they are all intrinsically empirical, despite some rational considerations explaining their structure. The flume experiments were made under controlled conditions, with more or less uniform flow.

A great number of bed-load formulae are in use today and are described in textbooks and manuals on sediment transport. The results of calculations obtained by these formulae may differ significantly. The choice of the formula depends on tradition or personal preference. In general, the following formulae are most widely used (Bruk, 1973):

In Western Europe - Meyer-Peter-Muller (with modifications), Engelund-Hansen, Bagnold

In the U.S.A. - the Einstein or Einstein-Brown formula, Laursen, Toffaleti, Engelund-Hansen

In the U.S.S.R. - Karaushev, Goncharov, Shamov, Levi

New formulae usually fit the existing flume data better because more data have been used for their development than for the older ones. The fit to empirical points remains, however, rather poor. Figure 5.9 after de Vries (1982), shows the probability distribution of the prediction of sediment transport by two formulae (Engelund-Hansen, and Ackers-White) by comparing predicted and measured sediment transport for laboratory flume experiments. In natural rivers the margin of error is even larger, though control is difficult since measured rates of bed load transport are poorly known.

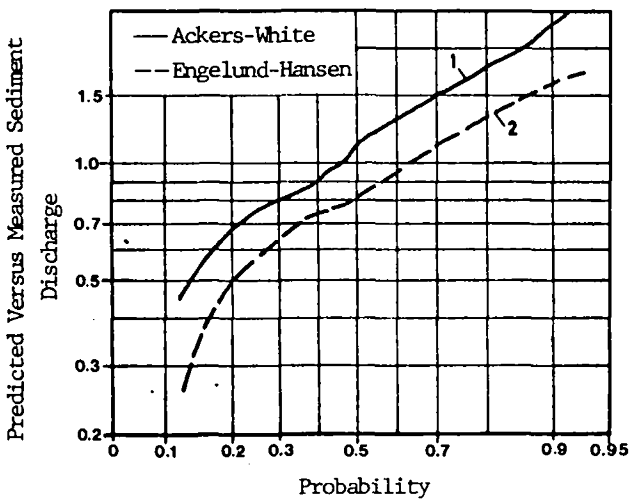


Figure 5.9 Probability distribution of the ratio predicted versus measured sediment discharges (de Vries, 1982)

The large error of prediction on bed-load transport makes the introduction of more sophisticated calculation methods illusory (nonuniform or even unsteady sediment transport).

Suspended Load

The calculation of suspended load in the one-dimensional model is based on formulae which express the capacity of the flow to transport a certain amount of particles of a given size in suspension. The actual load transported will equal the transport capacity only if there is sufficient supply of sediment particles available, either arriving from upstream reaches or picked up from the bed. Figure 5.10 illustrates the behaviour of suspended particles to changing flow conditions (after Rossinski and Kuzmin [1968] and Miloradov [1974]). Sedimentation occurs when the sediment content exceeds an upper limit, marked by the upper line on the diagram; erosion of the bed will occur when the actual sediment content is below a certain limit (the lower line on the diagram). Between these two limits, the flow can carry any amount of sediment in suspension.

The determination of the transport capacity is based on empirical information, although there is some theorétical background which explains the mechanism of sediment transport (Bruk, 1980). The use of the Rossinski-Kuzmin formula is recommended by Miloradov and Bruk (1968, 1979), since it unites all the measusrable parameters of the flow and is, thus, very apt for calibration on the rivers in question.

Total Load

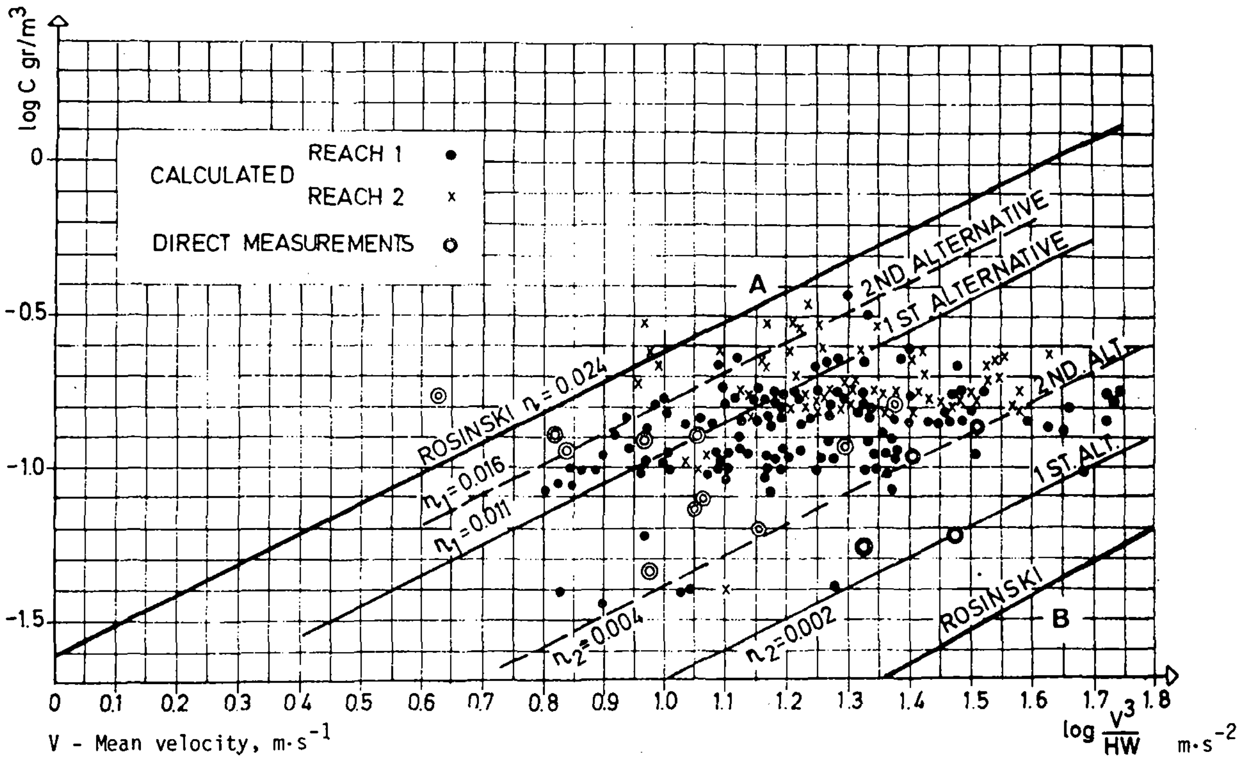
The sum of bed- and suspended load is sometimes called "the total load" and formulae have been proposed to calculate its magnitude (Einstein, Toffaleti - see, for instance Simons and Senturk, 1977). The assessment of the suspended load is in these formulae based on the integration of the vertical distribution of concentration, considering that the value of the concentration at bed level can be derived from the bed-load transport. The total load formulae unite the properties of both the bed-load and suspended load formulae and are entirely empirical despite the theoretical concepts which may be used for their development.

Density of Sediment Deposits

Since the calculations of sediment balance are made in mass units and the channel modifications are expressed in volume, the density of the deposits is a most important magnitude, on which the prediction of reservoir sedimentation depends. Whereas, for coarse sediments (gravel and sand) fairly good estimates of the density in deposits can be made by making use of the data from reference books (e.g., ASCE 1975); the estimated values for fine and very fine sediment may vary considerably.

Densities approaching unity have been measured for deposited silt and clay in many reservoirs. Proposals have been made to determine the density on the basis of the settling velocity of the particles, mineralogy, age of the deposits, reservoir operation, etc.; but no conclusive answer has been found so far. The practical way of surmounting this difficulty is to make the best possible evaluation of the expected densities from the available data and to define the limits of possible variations which may be used in the sensitivity analysis.

In case of the application of the model to existing reservoirs for the sake of prediction of future developments, a direct determination of the density of the deposits by field measurements is necessary. Progress has been made in the investigation methods as reported in the chapter on measurement techniques



V - Mean velocity, $m \cdot s^{-1}$

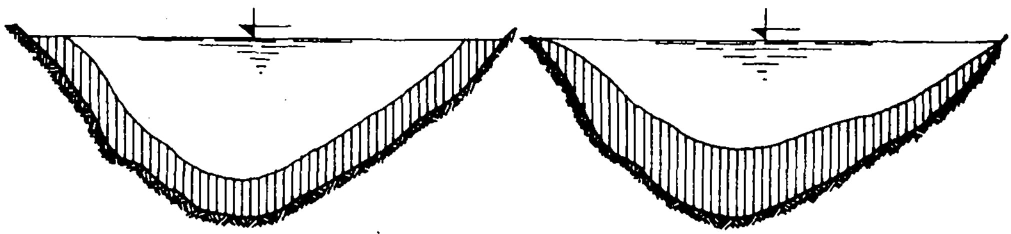
H - Mean depth, m

W - Mean settling velocity, $m \cdot s^{-1}$

Figure 5.10 Suspended load concentration correlated with hydraulic parameters, after Rossinski and Kuzmin (prepared by Miloradov and Varga, 1974)

Modification of Reservoir Topography

All calculations being one-dimensional, the position of the deposits within the cross-section of the reservoir remains undefined. Different assumptions can be made, but none of them is fully justified. Uniform spreading of the deposits across the reservoir floor is a simple assumption, but hardly acceptable (Figure 5.11.a); a more plausible situation occurs when the sediment is assumed to settle proportionally to the depth of the water (Figure 5.11.b). If there is branching of the channel, it is most likely that the shallower branch will become obstructed first, etc. In important cases, scale models can be used to predict the modification of channel geometry.



a b

Figure 5.11. Assumptions of channel modification in reservoirs
(a) - Uniform distribution of deposits
(b) - Thickness of deposit proportional to channel depth

5.3. PREDICTION OF STORAGE RECOVERY AND PRESERVATION

5.3.1 Unsteady flow model of Reservoir Emptying and Flushing

The approximation of the movement of water and sediment by a sequence of steady state situations is not effective when the changes in the flow conditions are rapid. In such cases the complete equations of Saint-Venant have to be used, including the inertial terms which could be neglected in the reservoir sedimentation studies.

Such an unsteady situation occurs in the case of emptying the reservoir and flushing the deposits using the water stored in it. This is a practice which may be invoked when some water in the reservoir remains unused or is deliberately sacrificed to the purpose of recovering a part of the lost storage. A mathematical treatment of the problem has been developed and applied to the Sefid Roud Reservoir of Iran (Cavor and Slavić, 1983).

In addition to the Saint-Venant's equations, describing the unsteady flow of water, the movement of sediment is described by an equation (describing heat or mass transport in a channel), integrated and averaged over the cross section (Jirka, 1975):

$$\frac{\partial \bar{c}}{\partial t} + \frac{\partial}{\partial x} \bar{u} \bar{c} + \frac{\partial}{\partial x} \overline{u^*c^*} = \frac{\partial}{\partial x} (K_T \frac{\partial \bar{c}}{\partial x}) + Q_{ss} \qquad (5.3.1)$$

where:

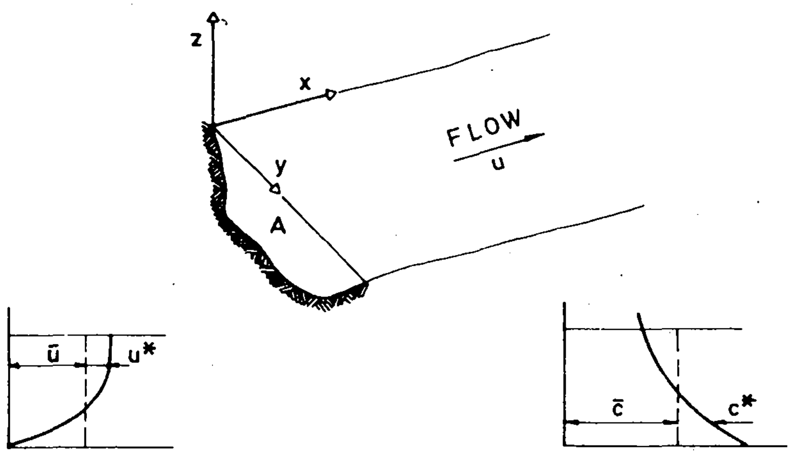
$\bar{c}$ - sectional average concentration

$\bar{u}$ - sectional average velocity in the direction of flow, m/s

u^*, c^* - systematic variations of c and u from the sectional average (see Figure 5.12)

K_T - turbulent exchange coefficient

Q_{ss} - a spatially averaged sink or source term.



Velocity profile

Concentration profile

Figure 5.12 Definition sketch for the equation of longitudinal dispersion

In using this equation, it is assumed that there is an instantaneous response of sediment transport to the changing velocities during the unsteady process. The covariation $\overline{u^*c^*}$ appears as a consequence of the systematic deviation of both velocity and concentration from the spatial mean, and is usually expressed by analogy with the turbulent flux $\overline{u'\,c'}$ as

$$\overline{u^*c^*} = - E_L \frac{\partial \bar{c}}{\partial x} \qquad (5.3.2)$$

where E_L is the longitudinal dispersion coefficient. The value of this coefficient, and thus of the dispersive flux $\overline{u^*c^*}$ varies within considerable limits (Fisher, 1969), and can be estimated for natural channels by calibration. The coefficient of diffusion K_T is usually smaller and the two coefficients can be combined in the calibration.

The response of the silt deposits is expressed by the term Q_{ss}. Its value is estimated by the instantaneous change of the transport capacity of the flow:

$$Q_{ss} = \Psi_{1,2} \frac{\partial c_T}{\partial t} \tag{5.3.3}$$

where the transport capacity C_T can be determined by a suitable formula. In the particular case, the Rossinski-Kuzmin formula (Rossinski and Kuzmin, 1964), was used; in principle, however, any formula could be used here, provided it has been found reliable by calibration or verification through field investigations.

In Equation 5.3.3, Ψ_1 is a coefficient for resuspension, and Ψ_2 of sedimentation. Both have to be determined by calibration (values between 0.20 and 0.35 were found for Ψ_1 and between 0.05 and 0.15 for Ψ_2).

The system of equations just described contains several empirical coefficients which have to be determined by calibration. In the case of Sefid Roud Reservoir, observations were available for two flushing operations (1980 and 1981): the input discharges of water and sediment concentrations, the variation of the reservoir level, and the released discharges and concentrations were measured during the two operations.

Once calibrated, the model can be used to predict the effects of flushing. By examining different possible patterns of operating the reservoir during the flushing, the most efficient of the patterns can be selected.

5.3.2 Retrogressive Erosion

For the special case of retrogressive erosion, Fan and Jiang (1980), have proposed a simplified calculation method which has proven satisfactory in the case of the Sanmenxia reservoir, as reported by the authors. The method is based on the balance between the eroded masses of deposit and the inflow and outflow of sediment (Figure 5.13).

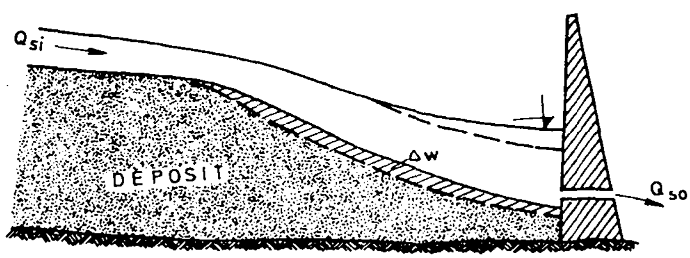


Figure 5.13 Definition sketch for retrogressive erosion
ΔW - Volume of eroded deposit through time Δt

$$\Delta W = (Q_{si} - Q_{so}) \Delta t \tag{5.3.4}$$

The notations are explained on the definition sketch. The inflow and out flow of sediment is expressed in weight (or mass).

The inflowing sediment Q_{si} is obtained from the inflow hydrograph. The outflowing sediment Q_{so} is determined by an empirical equation, obtained by measurements on the Sanmenxia reservoir:

$$Q_{so} = 3.5 \times 10^{-3} Q_o^{1.2} J^{1.8} \tag{5.3.5}$$

where Q_o is the outflow of water.

The eroded volume the authors defined by assuming a simplified geometry of scour, as shown on Figure 5.14:

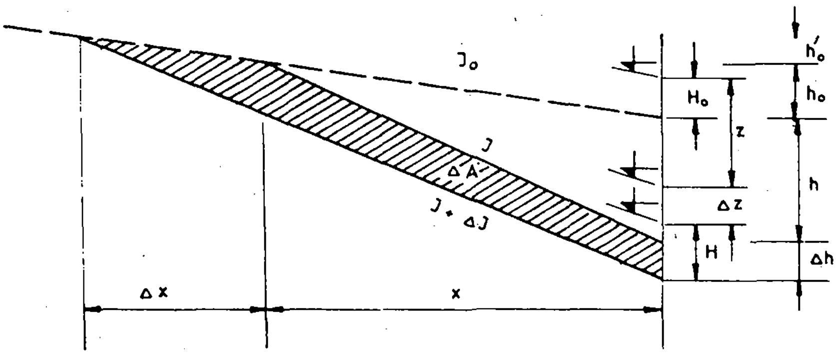


Figure 5.14 Calculation of retrogressive erosion after Fan and Jiang (1980)
H - depth of flow; z - lowering of water level; h - scour

The eroded volume is then obtained by:

$$dw = B\cdot\gamma\cdot dA = B\cdot\gamma\ 0.5\ [(x + dx)(h + dh) - x\cdot h] = 0.5(x\ dh + h\ dx)\cdot B\cdot\gamma$$

where B - width of the deposit, γ - unit weight (or mass).

After having expressed x and dx by the geometrical relationships which follow from Figure 5.14 (and allowing that the increment of slope dJ is small compared to the difference $(J - J_o)$, the authors have obtained the following relationship, which could be then used for integration:

$$0.5\cdot\gamma\cdot B \left[2h\ dh/(J-J_o) - h^2\ dJ/(J-J_o)^2\right] = (K\ Q_o^{1.2}\ J^{1.8} - Q_{si})dt \tag{5.3.6}$$

with the initial conditions:

$$J_{t=0} = J_o \qquad dJ/dt\Big|_{t=0} = J'_o$$

The computation was made by numerical integration:

$$\frac{dJ}{dt} = \frac{2(J-J_o)}{h} \cdot \frac{dh}{dt} - \frac{2}{B} \cdot \frac{(J-J_o)^2}{h^2} (K \cdot Q_o^{1.2} \; J^{1.8} - Q_{si}) \tag{5.3.7}$$

with:

$$J_{n+1} = J_n + \Delta t \left.\frac{dJ}{dt}\right|_{t=t_n} \qquad J_n = J_{t=t_n}$$

The method was applied with success by Chinese engineers; (Qian, 1982). It requires apparently a sound basis of field measurements and does not represent a universally applicable model of sediment removal from reservoirs.

5.3.3 Venting of Density Currents

A full mathematical description of the appearance, propagation, modification and outflow of density currents is still not possible at the present state of the knowledge. The basic equations are, however, well defined and by making use of some assumptions and semi-empirical approximations, the designer may arrive at reasonably fair predictions (Qian, 1982).

The equation of motion can be written in the following form (Fan, 1979):

$$u_2 \cdot \frac{\partial u_2}{\partial x} + \frac{\partial u_2}{\partial t} = - \frac{\rho_2 - \rho_1}{\rho_2} \cdot g \cdot \frac{\partial h_2}{\partial x} + \frac{\rho_2 - \rho_1}{\rho_2} \cdot g \cdot \frac{\partial (h+z)}{\partial x} - \frac{\tau}{\rho_2 h_2} \tag{5.3.8}$$

The notations are explained in Figure 5.15.

Thus, the equation of motion of the density current differs from the St. Venant's equation for unsteady flow in open channels by the reduction of the gravity constant, by multiplying it with $(\rho_2 - \rho_1)/\rho_2$.

The continuity equation, in which gravity does not appear, remains unaltered:

$$\frac{\partial h}{\partial t} + u \cdot \frac{\partial z}{\partial x} + h \cdot \frac{\partial u}{\partial x} = u_{xx} \tag{5.3.9}$$

The term on the right-hand side takes care of the mass exchange between the density current and the top layer.

The main difference between the St. Venant's equation and the above ones is that the reduced gravity is not a constant, but depends on the density difference between the turbid underflow and the top layer in the reservoir. The density of the underflow is, however, variable, depending on the sediment content: sediment particles may settle partly along the path of the current, but

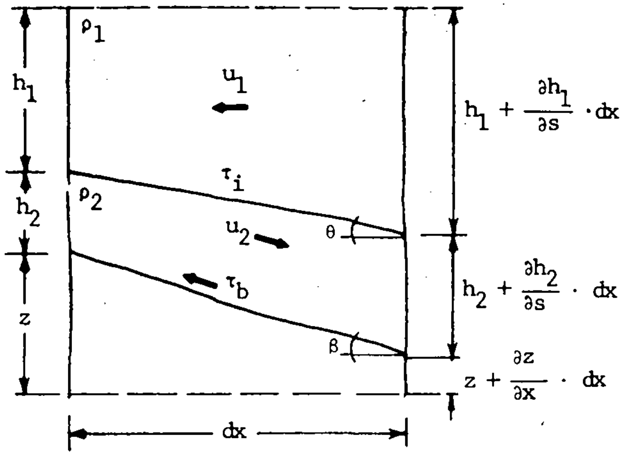


Figure 5.15. Definition sketch for the equations of density currents

can also be picked up by it from the bed. Thus, a transport equation of the form of equation (5.3.1) should be added to the two preceding ones.

The solution of the above set of equations requires the definition of boundary conditions and an estimate of the unknown terms (channel resistance, momentum-exchange, mass exchange, diffusion, etc.).

Upstream Boundary

The upstream boundary is defined by the cross section in which the muddy river flow plunges below the clear and stagnant water (Figure 5.16).

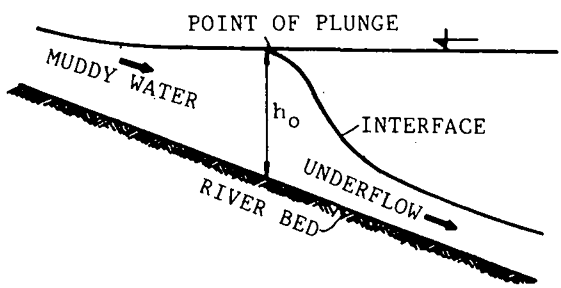


Figure 5.16. Point of plunge of a density current (Fan, 1960)

It is evident from the diagram, that the velocity decreases as the depth increases in the transition zone from the open channel flow to the density current; the velocity becomes minimal and the depth of the muddy flow approaches its maximum at the spot of plunge (Fan, 1960). Thus, the specific energy of the flow has a minimum value at this point, i.e., the Froude number has the value of unity:

$$E = h + \frac{u_2^2}{\frac{\rho_2 - \rho_1}{\rho_2} 2gh} \longrightarrow \text{Min} \qquad \frac{u_{20}}{\sqrt{\frac{\rho_2 - \rho_1}{\rho_2} gh_o}} = 1 \tag{5.3.10}$$

Actually, the real critical value of the densimetric Froude number is less than unity, and experimental evidence shows that it is equal to:

$$\frac{u_o}{\sqrt{\frac{\Delta\rho}{\rho_2} \cdot gh_o}} \approx 0.78 \tag{5.3.11}$$

The difference is caused by the curvature of the streamlines.

In consequence, the flow of the density layer is not controlled from the downstream end; its development depends upon the bed slope and channel characteristics.

Downstream Boundary

The flow at the downstream boundary is controlled by the bottom outlets of the dam. Different ratios of clear and turbid water may enter the outlets, depending upon the relative positions of the interface and the outlets and on the discharge released.

The classical solution of Craya (1949), confirmed and widely used by many others, concerns two-layered stratification and unconfined depth. The densities in both layers are constant. For a slot or an orifice, critical values of the densimetric Froude number can be defined:

slot: $$F_s = \frac{q}{d\sqrt{g'd}} = 1.52 \tag{5.3.12}$$

orifice: $$F_o = \frac{Q}{d^2\sqrt{g'd}} = 2.52 \tag{5.3.13}$$

Here, as before, $g' = \frac{\Delta\rho}{\rho_1} \cdot g$

and the notations are explained in Figure 5.17.

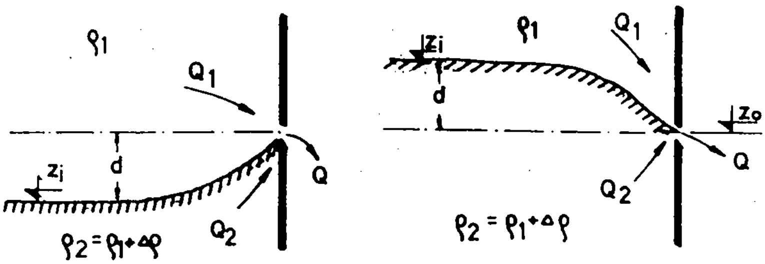


Figure 5.17 Outflow of a two-layer fluid through an orifice or slot (after Craya, 1949)

The meaning of these critical Froude numbers is the following: when the interface is below the orifice (or slot), no discharge from the lower layer will be withdrawn ($Q_2 = 0$) unless the Froude number becomes higher than the critical value (Figure 5.17a); and vice versa, when the interface is above the orifice (or slot), no discharge from the upper layer will enter the outlets ($Q_1 = 0$), unless the Froude number becomes higher than the critical value:

for	$z_i < z_o$	$Q_2 = 0$	for	$F \leq F_c$
	$z_i > z_o$	$Q_1 = 0$	for	$F \leq F_c$

The solution of Craya has been extended to higher Froude numbers, when both layers are affected. Wood (1978) gives a parametric solution of the ratio Q_1/Q as a function of the Froude number:*

$$\frac{Q_1}{Q_2} = \left(\frac{1-\eta}{1+\eta}\right)^{3/2} \left(\frac{2+\eta}{2-\eta}\right)^{1/2} \tag{5.3.14}$$

$$F = \frac{Q}{(g'd^5)^{0.5}} = 2.54 \left(\frac{\eta+1}{2}\right)^{3/2} \frac{(2-\eta)^{1/2}}{\eta^{5/2}} \left(1+\frac{Q_1}{Q_2}\right) \tag{5.3.16}$$

$$\frac{Q_1}{Q} = \frac{Q_1/Q_2}{1 + Q_1/Q_2} \tag{5.3.16}$$

Here, η is a dummy variable.

The solution of these equations is shown in Figure 5.18. It can be seen that for $Q_1 = 0$, $\eta = 1$ and $F = 2.54$ in accord with the critical value of Crya.

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*Rearranged by Kupusovic (1982), $F_{crit} = 2.54$

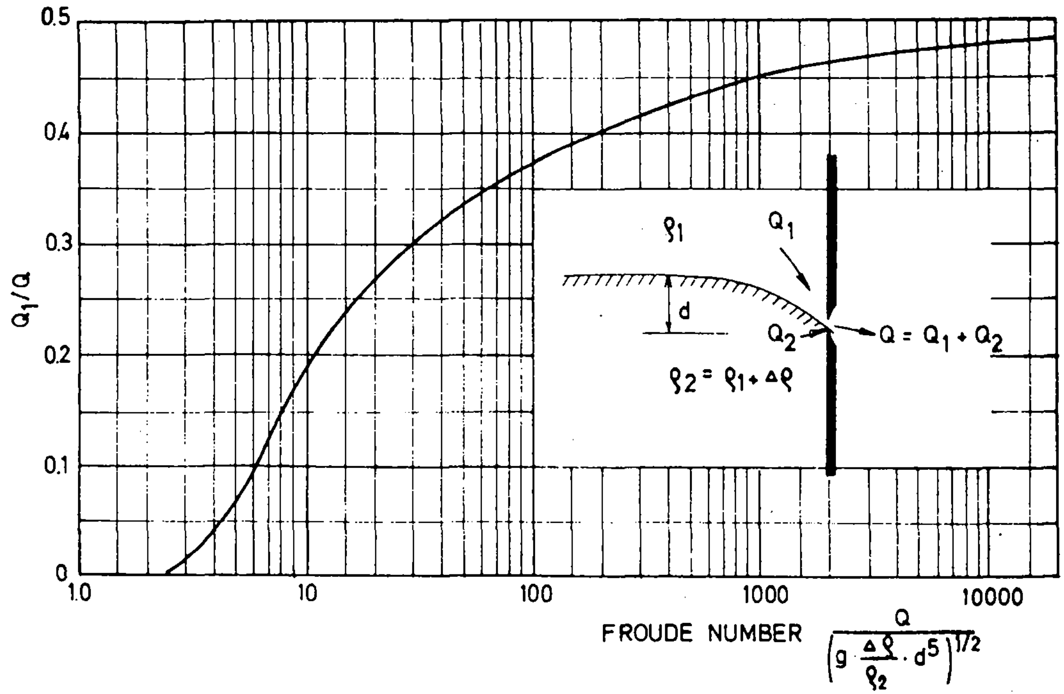

Figure 5.18 Solution of the outflow of a two-layer fluid after Wood, 1978 (reinterpreted by Kupusovic, 1982)

With increasing Froude number, the ratio Q_1/Q tends to 0.5, i.e., $Q_1 = Q_2$, which is the limit value as long as the interface is above or at the level of the orifice. Departing from the criteria of Craya, Fan (1962) proposed a solution, supported by laboratory flume experiments. After having made a few reasonable assumptions, he derived the following relationships for a two layer fluid, with constant densities ρ_1 and ρ_2:

$$\text{slot:} \quad \frac{\rho_2 - \rho_o}{\rho_2 - \rho_1} = \frac{1}{2}\left[1 + K_s\,(z_i - z_o)\left(\frac{g'}{q^2}\right)^{1/3}\right] \tag{5.3.17}$$

$$\text{orifice:} \quad \frac{\rho_2 - \rho_o}{\rho_2 - \rho_1} = \frac{1}{2}\left[1 + K_o\,(z_i - z_o)\left(\frac{g'}{Q^2}\right)^{1/5}\right] \tag{5.3.18}$$

where ρ_o is the outflow density:

$$\rho_o = \frac{Q_1 \cdot \rho_1 + Q_2 \cdot \rho_2}{Q_1 + Q_2}$$

$\rho_o = \rho_2$ when the lower layer only is affected ($Q = Q_2$) and

$\rho_o = \rho_1$ when the upper layer only is affected ($Q = Q_1$).

Instead of density, the volumetric concentration can be used for a sediment-carrying flow:

$$\frac{\rho_2 - \rho_o}{\rho_2 - \rho_1} = \frac{c - c_o}{c} \tag{5.3.19}$$

where c_o is the outflow concentration. The upper layer is here assumed to have $c = 0$.

Fan (1960) has found the following values for the constants K_s and K_o

$$\text{for } z_i < z_o \qquad 0 < \frac{\rho_2 - \rho_o}{\rho_2 - \rho_1} < 0.5 \qquad \left\{\begin{array}{l} K_s = 1.55 \\ K_o = 1.67 \end{array}\right. \tag{5.3.20}$$

$$\text{for } z_i > z_o \qquad 0.5 < \frac{\rho_2 - \rho_o}{\rho_2 - \rho_1} < 1 \qquad K_s = K_o = 1.33 \tag{5.3.21}$$

The outflow concentration or density can easily be estimated from the above relationships, for any position of the interface, density difference and discharge.

The above formulae are valid only for constant densities in both layers. Since in a sediment-laden turbid current the density increases with depth, the expressions (5.3.17) and (5.3.18) need some adjustment. Fan (1960) made the following proposal:

slot:
$$\frac{\rho_2 - \rho_o}{\rho_2 - \rho_1} = \frac{1}{2}\left[1 + K_s\,(z_i - z_o)\left(\frac{g'}{q^2}\right)^{1/3} + \delta_s\right] \tag{5.3.22}$$

orifice:
$$\frac{\rho_2 - \rho_o}{\rho_2 - \rho_1} = \frac{1}{2}\left[1 + K_o\,(z_i - z_o)\left(\frac{g'}{Q^2}\right)^{1/5} + \delta_o\right] \tag{5.3.23}$$

Different values of the constants were given for different depths of the orifice (or slot), obtained by flume experiments.

For $z_o = 0$, i.e., for an orifice located immediately above the bed, the following relationship was proposed:

$$\frac{\rho_2 - \rho_o}{\rho_2 - \rho_1} = 1 + K_o \cdot z_i \left(\frac{g'}{Q^2}\right)^{1/5} \tag{5.3.24}$$

The value of K_o obtained by the flume experiments was 0.47, while field measurements on the Kuanting reservoir of China have given the value 0.415.

Other authors have treated linear stratification in a different way. Since the governing factor of stratification is the density gradient $d\rho/dz$, the thickness of the layer affected by the outflow is given by expressions of the following form:

for a slot:
$$\delta_s = K_s \left(\frac{q}{\sqrt{-\frac{g}{\langle\rho\rangle}\cdot\frac{d\rho}{dz}}}\right)^{1/2} \tag{5.3.25}$$

for an orifice:
$$\delta_o = K_o \left(\frac{Q}{\sqrt{-\frac{g}{\langle\rho\rangle}\cdot\frac{d\rho}{dz}}}\right)^{1/3} \tag{5.3.26}$$

where $\langle\rho\rangle$ is the mean density of the fluid, and K_s and K_o are constants (of the order of 2.0 and 0.9, respectively).

In a recent study, Kupusovic (1982) analyzed the case of a two layer stratification, in which the density of the lower layer increases linearly with depth. By making use of the relations of Craya, equations (5.3.12) and (5.3.13) and of equations (5.3.25) and (5.3.26), two critical discharges could be defined:

for a slot:
$$q_c = F_s \; d\sqrt{g'd} \tag{5.3.27}$$

$$q_* = \frac{4d^2}{K_s^{\,2}} \sqrt{-\frac{g^*}{<\rho>} \cdot \frac{d\rho}{dz}} \tag{5.3.28}$$

for an orifice:

$$Q_c = F_o \; d^2 \sqrt{g'd} \tag{5.3.29}$$

$$Q_* = \frac{8d^3}{K_o} \cdot \sqrt{-\frac{g}{<\rho>} \cdot \frac{d\rho}{dz}} \qquad d = \frac{\delta}{2} \tag{5.3.30}$$

It is evident that

$$q_c < q_* \quad \text{and} \quad Q_c < Q_*$$

The following three cases may occur:

1. The actual discharge released is smaller than q_* or Q_*:

$$q < q_* \qquad Q < Q_*$$

Water is withdrawn only from a part of the lower layer, which is linearly stratified.* The thickness of this part of the layer can be estimated from Equations (5.3.25) or (5.3.26).

2. The actual discharge is between the two critical values:

$$q_c < q < q_* \qquad Q_c < Q < Q_*$$

The entire depth of the lower layer is then affected, but the upper layer remains undisturbed.

3. The actual discharge is greater than the higher of the two critical discharges:

$$q > q_c \qquad Q > Q_c$$

Both layers are then affected and an estimate of the outflow density or the ratio Q_1/Q can be made by the appropriate formulae.

The Channel Resistance

The last term on the right-hand side of Equation (5.3.8) represents the resistance to the flow, offered by the bed friction, friction at the interface and also by the momentum exchange across the interface. Although it is possible to separate these factors theoretically, for practical purposes a bulk value can be assigned to the overall resistance, in the conventional form of a resistance

*The density of the lower layer can also be assumed to be constant:

$$\rho_e = \rho_1 + \Delta\rho_e = \rho_1 + \Delta\rho + \alpha^2 \Delta\rho$$

where $\alpha^2 \approx 1/3$ (after Kuposovic, 1982).

coefficient. According to Fan (1960, 1961) good results can be obtained by using an appropriate value of the Darcy-Weissbach friction coefficient, from 0.025 to 0.03.

Transport Capacity of the Density Current

Coarse particles are very soon deposited by the current after entering the reservoir. The underflow thus contains only fine particles, with a mean diameter of the order of 0.002 to 0.003 millimeters, and D_{90} of the order of 0.01 to 0.1 millimeters. The sediment content, however, changes along the course of the underflow: decreasing velocity causes deposition of particles, and increase of velocity (caused by channel geometry) leads to resuspension of sediment particles by the current.

The transport capacity of the density current can in principle be estimated by the usual methods for sediment carrying streams, i.e., by semi-empirical formulae described in the preceding paragraphs. Based on observations on reservoirs in China (Guanting and Sanmenxia), Fan (1981) proposed a diagram linking its capacity to carry sediment particles of a given size (expressed by D_{90}) with the flow velocity (Figure 5.19).

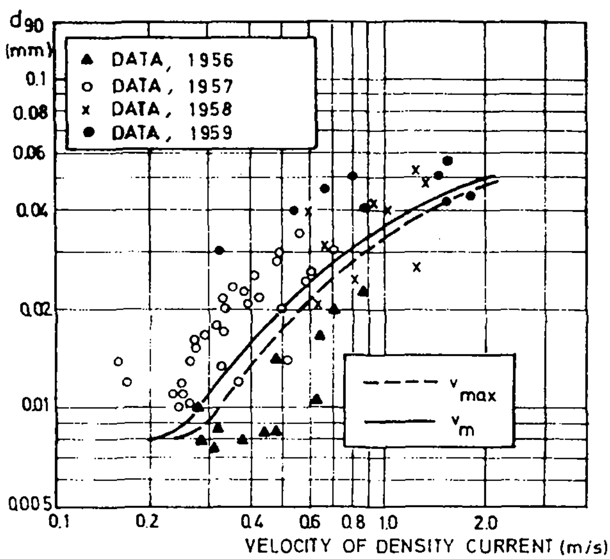

Figure 5.19 Sediment particle size carried by a density flow as a function of flow velocity (after Fan et al., 1962)

Fan thus assumes that the current can carry only the part of the grains which have a diameter smaller than the given values, and recalculates the sediment concentration from the grain size distribution of the entering sediment.

Calculation Methods

The above described set of equations, together with appropriately defined boundary conditions and constants, can be used for the development of mathematical models using numerical methods to solve the differential equations of unsteady flow. The many assumptions and simplifications, which have to be made in order to develop such a model, require calibration by field measurements. Very few data exist, however, and this is probably why no known attempts have been made so far to develop complete models despite the availability of suitable computational facilities.

Fan et al. (1962) have proposed a simplified calculation method, which is based on the division of the reservoir into reaches, and the application of uniform flow equations for each reach. The calculation consists of 18 successive steps, which are illustrated by Fan on two examples: for the well-known case of Lake Mead in the U.S.A. (1935) and for the Guanting Reservoir in China (1954).

5.4 PREDICTION OF DOWNSTREAM EROSION

Degradation of river beds downstream from dams is a consequence of the retention of sediments in the reservoirs. The transport capacity of the flow is not increased as compared to the natural regime of the river, but the sediment-free flow is able to pick up particles from the bed and carry them further downstream in the form of bed load or suspended load. Erosion of the river bed continues until a new equilibrium is established between the transport capacity of the flow and the resistance of the river channel (Figure 5.20).

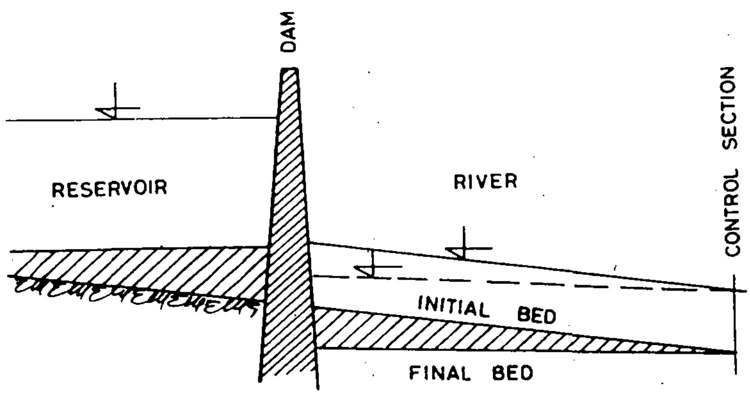

Figure 5.20 Degradation of the river bed downstream from a dam

Bed degradation can have many consequences: it may cause bank erosion, underscouring embankments, river structures, bridge piers and abutments; deteriorate the navigable waterway, including river harbors and structures in ports, etc. Prediction of downstream erosion is therefore an important task of the reservoir design.

A considerable number of calculation methods have been developed in recent years for the prediction of erosion or channel degradation downstream from dams. Most of these were based on theoretical considerations and laboratory flume experiments and only very few claim confirmation of the predictions by field measurements (Hammad, 1972).

Empirical formulae, based on field experience, were also proposed (e.g., Priest and Shindala, 1969). They lack generality, however, and will not be discussed here. From the point of view of mathematical modelling, there is no difference between the sedimentation and degradation processes. In principle, the models are based on the balance of sediment, reach by reach, the deficit of the balance being covered by erosion from the river bed (or vice versa, the excess being deposited in the reservoir or river channel). The equations of flow and sediment transport were discussed in paragraph 5.2 and there is no need to repeat them here. Reservoir sedimentation models could also be used for prediction of downstream erosion, provided that appropriate factors are included in the model. For instance, the program package HEC-6 is equally recommended for the prediction of both aggradation and degradation (Thomas and Prasuhn, 1977). Many attempts have been made to solve the degradation equations analytically, but they are of little interest for practical purposes because of the severe simplifications and unproved assumptions which have to be introduced. The irregular river channel, variable flow and sediment characteristics justify numerical methods only, as in reservoir sedimentation problems.

Some important specific features of the models for the prediction of downstream erosion merit attention.

Erodibility of the River Bed

If the river is composed of uniform sand, erosion continues until the appearance of rock or other resistant material below the layers of sand, or when the flow velocity becomes reduced under the influence of downstream controls. If the bed is composed of graded sand or gravel, the finer particles will be washed away first, and the bottom of the river will gradually be covered up by the remaining coarser particles. This is the so-called armouring of the river bed, which is in most cases an important limiting factor of erosion. If the bed is composed of cohesive soil, its erodibility depends on the amount and type of the clay in the soil.

The incipient motion of loose alluvial particles can be expressed either in terms of a critical velocity or a critical shear force. Both approaches are equally justified and may be used in the model. An excellent analysis of critical conditions was given recently in the book of Stelczer (1980). In the applications, the critical conditions are determined by means of empirical diagrams or formulae (e.g., the Shields diagram for critical shear or the Hjulström diagram for critical velocities), originally developed for uniform grains. For nonuniform materials, the use of the well-known formula of Egiazaroff (1965) is recommended on the ground of extensive laboratory research (Rakoczi, 1975). Incipient motion can easily be tested on samples of bed material in laboratory flumes.

In the case of cohesive soils, Ariathurai (1980) recommended specific laboratory tests on samples taken from the river bed. The samples were tested in a rotating cylinder apparatus described by Kandiah (1974). The chemical properties, such as the cation exchange capacity (CEC) of the samples, were also investigated to obtain a measure of the type and the amount of clay in the soil. Ariathurai has obtained good correlation between CEC and the critical shear stress.

Density of Bed Material

Since sediment transport is calculated in mass (or weight), the deficit (or excess) has to be converted into volume in order to predict channel modifications. The bulk density of the bed material is thus a very significant parameter of the calculations. There exist empirical formulae or diagrams which correlate the bulk density with grain size distribution and other characteristics of the material, as mentioned in paragraph 5.2. For fine materials the prediction of density is not reliable and the in-situ density should be determined, whenever possible, on undisturbed samples. Methods of taking samples are described in Chapter 3 of this report.

Evaluation of Armoring and Stability of the Armor Layer

Since armoring develops by the removal of the finer particles from the bed, the simplest way of estimating the composition of the armor layer is by taking off from the initial grain size distribution all those particles which have critical velocity (or shear) lower than the actual velocity (or shear) of the flow. This method is illustrated in Figure 5.21.

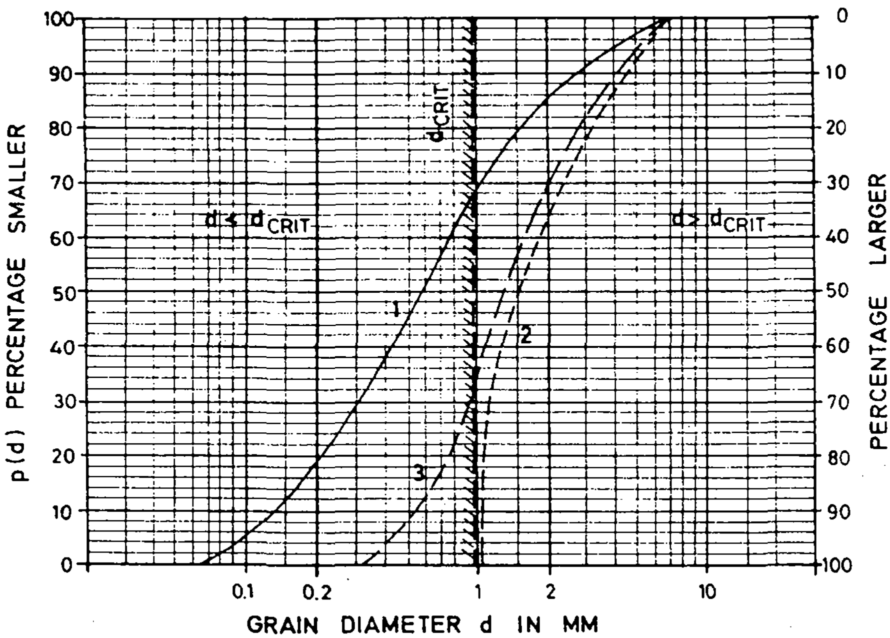

Figure 5.21 Armoring of river bed downstream from a dam
(1) - Original bed material
(2) - Armored bed obtained by elimination of $d < d_{crit}$
(3) - Armored bed obtained by the method of Gessler

This simple model was improved by Gessler (1970, 1971) who considered that in turbulent flow both the velocity and the shear stress fluctuate and in consequence, there is no sharp limit between mobile and immobile particles. Gessler assumed that the shear stress is distributed normally around its mean value (with standard deviation equal 0.57, obtained empirically) and, therefore, the probability of the shear being lower than a critical value is given by the integral:

$$P\ (\bar{\tau} < \tau_c) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{z} e^{-\frac{z^2}{2}}\, dz \tag{5.4.1}$$

where

$$z = \frac{(\tau_c/\bar{\tau}) - 1}{\sigma} = \frac{\tau_c - \bar{\tau}}{\bar{\tau} \cdot \sigma} \tag{5.4.2}$$

The above relationship is illustrated in Figure 5.22.

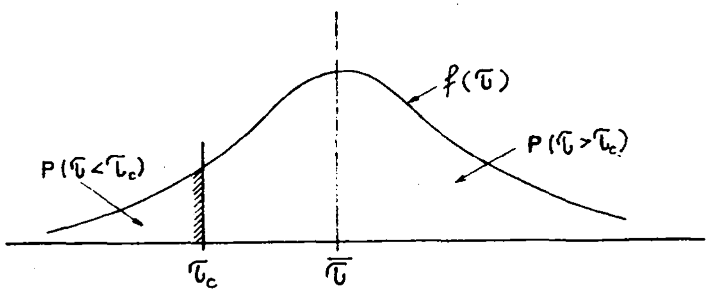

Figure 5.22 Probability of distribution of shear stress (after Gessler, 1971)

The probability of $P\ (\bar{\tau} < \tau_c)$ represents also the probability of grains of diameters larger than D_{crit} not being moved by the flow. In consequence, the armor layer will also contain particles having smaller diameters than D_{crit}. Figure 5.21 illustrates the generation of a new grain size distribution curve after the method of Gessler.

The method of Gessler has been widely used also by other authors, though some doubts were expressed about the general validity of the normal distribution of shear stresses, and particularly, having a standard deviation of 0.57. Experimental studies in the laboratory and in the field are, therefore, highly recommended.

In an experimental study, Little and Mayer, 1976, proposed several empirical formulae for the determination of the grain size distribution of armor layers. These are valid, of course, for the investigated materials only. Rakoczi (1977) had recommended the analysis of grain size distributions of self-paved stretches of rivers in order to examine the development of the armoring process.

Armoring can develop in nonhomogenous materials only. After Knoroz (quoted by Lapshenkov, 1979), the necessary condition for armoring is given by the ratio:

$$\frac{D_{90}}{D_{50}} > 5 \tag{5.4.3}$$

A more complex formula of the same author is quoted by Simons and Sentürk (1977).

The sediment particles in the armor layer are placed so that they offer minimum resistance to the flow. The armor layer can thus be destroyed only by shear forces (or velocities) considerably higher than the ones which contributed to its development. After Rakoczi (1977) the break-up of the self-pavement (or armor layer) can be expected when the shear stress exceeds the critical value which corresponds to the D_{95} of the original grain size distribution

Gessler (1971) defined a mean value of the probability that a nonhomogenous mixture of sediments will stay stable by making use of the hypothesis of normal distribution of shear. Hence,

$$\bar{P}_A = \frac{\int_{D_{min}}^{D_{max}} P_A^2 \cdot f \cdot dD}{\int_{D_{min}}^{D_{max}} P_A \cdot f \cdot dD} \cong \frac{\sum P_A^2 \cdot f \cdot \Delta f}{\sum P_A \cdot f \cdot \Delta f} \tag{5.4.4}$$

where f (D) is the grain size distribution and P_A is obtained by Equations 5.4.1 and 5.4.2.

The bed would be absolutely stable for $\bar{P}_A = 1$, which is possible only for $\bar{\tau} = 0$. On the other hand, the material would not resist at all if $\bar{P}_A = 0$, which would be the case of infinite shear. Between these two extremes, Gessler considered the bed to be stable, if P_A is higher than 0.5. Other authors (Thomas and Prasuhn 1977), however, preferred a higher value of this parameter: they considered that the armor layer is stable only if $\bar{P}_A$ is higher than 0.65.

In important cases, the question of stability of the armor layer can be settled by special laboratory flume tests of the bed material samples.

Stability of the River Banks

An important consequence of the bed degradation is the underscouring of river banks. The stability of these has to be checked by means of the methods used in soil mechanics. It is important to take undisturbed soil samples of the banks or to conduct in-situ tests of bank stability in order to estimate the effects of bed degradation.

Downstream Control

The degradation of the river bed is limited at the downstream end by a control section, which may be a natural sill, rock or simply the backwater from the mouth of the river into another river or the sea. Man-made structures can also provide the control, such as bottom sills, weirs, barrages or dams. The calculation of bed degradation begins usually from the downstream control section (as the flow is mostly subcritical) and proceeds upwards, up to the dam. In Russian literature, the downstream control is called "the basis of erosion."

Characteristics of the Flow Released from the Reservoir

The regime of the flow which is released from the reservoir will, in general, be rather different from the natural regime of the river. The flood discharges will be reduced considerably, depending on the capacity of the reservoir, the flood hydrograph and the operation of the reservoir. Since the movement of sediments is most intensive during the floods, the transport capacity of the river may be reduced considerably as compared to its values before the dam was built. This is a significant fact which limits the erosion to some extent.

On the other hand, if a power plant is operated from the reservoir, the released flow may be highly variable, especially if peak power is generated. This may increase the transport capacity as compared to the natural regime of flow. It should be noted, that since the flow is highly unsteady, the inertial terms of the Saint Venant's equations should not be neglected as is usually the case in river hydraulics.

Sediment Content of the Flow Released from the Reservoir

Since the erosion depends on the saturation of the flow downstream from the dam, it is very important to have reliable data on the sediment content of the released flow. In the design stage, this can be obtained only by calculations of the sedimentation in the reservoir. Both the concentration and the grain sizes of the sediments are important. It is obvious, that flushing of sediments or bypassing heavily sediment laden flows to the river downstream from the dam have a favorable effect on limiting the degradation of the river bed.

Bed Roughness

The coarsening of the bed material is sometimes taken into account in the calculations, by suitable modifications of the roughness coefficient (Egiazaroff, 1967). Since, however, the bed roughness is only one of the factors determining the resistance of the channel besides the channel irregularities, meanders, obstacles, etc., this seems to be in many cases of secondary importance only.

Channel Modifications

Just as in the case of reservoir sedimentation, the prediction of channel modifications by the one-dimensional models of bed degradation needs additional assumptions. Some of these were discussed already in paragraph 5.2 (Figures 5.11a and 5.11b). If more exact information is needed, the use of scale models can be recommended. Two-dimensional mathematical models are also applicable, at least for shorter river reaches.

Secondary Influences

In some cases, the degradation process may be influenced strongly by factors which are apparently of a secondary character. For instance, large amounts of sediment can be accumulated in the river bed downstream from the dam during its construction. Snishchenko (1977) reports a dam in the USSR where sediments accumulated during the construction of the dam had still not been removed by the flow after 23 years.

Road construction, work on tributaries, etc., sometimes supply large quantities of sediment to the river and moderate considerably the degradation of the river bed. On the other hand, excavation of gravel from the river bed for construction purposes may have an opposite effect.

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Chapter 6
Conclusions and Recommendations

The present report has been prepared for the benefit of planners and engineers, particularly in developing countries where water resources projects rely on reservoirs subject to sedimentation and its detrimental effects.

The report, however, is not a guide with detailed practical instructions on how to deal with the various aspects of reservoir sedimentation in all stages of the project; the preparation of such a guide is expected to be included in the 3rd Phase of the International Hydrological Program. This report could serve as a background to the outline of the guide.

The recommendations that conclude the report are thus addressed to those who investigate and study reservoir sedimentation problems, aiming at the improvement of the methods of planning, design and operation of reservoirs.

6.1 EVALUATION OF THE IMPACTS OF RESERVOIR SEDIMENTATION

Reservoirs are a vital part of every water resources development project, built or planned for important social and economic purposes. The construction of a reservoir greatly changes the physical conditions of the river and its basin, consequently wielding a far-reaching, economic and social impact on the environment.

6.1.1 Physical and Ecological Influences

The reservoir disturbs the sediment regime of the river: deposition of sediment occurs in the reservoir and upstream from it, and erosion of the river bed takes place downstream from the dam. The effects of sedimentation are manifold. It is recommended to study the possible impacts of sedimentation on the planned performance of the water resources project in the planning stage of the project, including the effects of sedimentation on the useful life of the reservoir; on the physical conditions up and downstream from the reservoir; on the ecology of the reservoir and downstream river reach.

6.1.2 Influences on Structural Safety

The effect of the expected sedimentation on the safety of the dam and appurtenant structures (spillways, gates, bottom outlets, etc.) should be taken into account in the design stage. Facilities should be provided to evacuate floating debris without obstructing the spillway and the gates. The concrete used for dams and other structures should resist chemical influences of water and sediments. Precautions should be taken to secure the bottom outlets against obstruction, wear and corrosion, enabling repairs in the case of damage.

6.1.3 Economic and Social Influences

The loss of storage by sedimentation affects the various users of the reservoir: the economic consequences of this should be evaluated and included in the socioeconomic studies by which the feasibility of the water resources project is justified. Neglect of sedimentation in the analysis may lead to failure of the project due to its falling short of the expected benefits or the necessity of costly remedial measures in the future.

6.2 UNDERSTANDING OF THE PHYSICAL PHENOMENA

Although the principal factors and the mechanism of reservoir sedimentation and related phenomena are fairly well understood, uncertainties remain with regards to the quantitative assessment of the process and its representation by mathematical or physical models. These uncertainties impair considerably the accuracy of predictions on which decisions on reservoir planning, design and operation depend.

6.2.1 Physics of Sedimentation

The apparition and influence of currents induced by wind and temperature differences on the turbidity currents in lakes and reservoirs should be better understood and calculation methods developed for quantitative prediction. Questions such as flocculation of fine sediments, density, and mobility of loose sediment deposits need further clarification.

6.2.2 Quality of Sediment Deposits

Information on the quality of sediment and sediment deposits is important for storage recovery practices, such as the flushing or dredging of the deposits. The ecological impact of these practices on downstream river reaches is, in many cases, the limiting factor; in other cases, the use of deposited sediments for land improvement or as building material can reduce the costs of these measures.

6.3 MEASUREMENTS AND RESERVOIR SURVEYS

The ultimate fate of all reservoirs is to become filled with sediment, except if a long-term usable capacity can be maintained by appropriate measures. In the planning phase, it is important to estimate the useful life of a reservoir - the useful life being the time before it becomes filled with sediments to the point of no longer functioning as designed. The rate of sediment accumulation in a reservoir depends upon the amount of sediment delivered to it by rivers and streams and the trap efficiency of the reservoir, which is a function of sediment size. All these things must, therefore, be considered in the planning stage. Consideration must be given to the individual reservoir, as well as the interdependence of all reservoirs and hydraulic structures in the basin.

6.3.1 Sediment Measurement

The methods of sediment measurement are still imperfect: bed-load cannot be measured accurately in natural rivers and streams, especially during high floods when most of the sediment moves. Suspended load can be measured with higher accuracy. The determination of grain size distribution of suspended sediment is essential for sediment calculations, but is often neglected in routine investigations.

Determination of channel resistance or roughness is also essential for sediment calculations. It is strongly recommended that it be determined by field measurement instead of using textbook references.

6.3.2 Reservoir Survey

Since the rate of sediment accumulation in reservoirs cannot be accurately predicted, it is always recommended that the volume and weight of sediment accumulated be determined during specific intervals of time in the course of the life of the reservoir. During the design phase of the project, it is necessary to make a plan and provision for the execution of these sediment surveys.

6.3.3 Surveying Techniques

The basic procedure for executing sediment surveys has changed little for many years, but there have been great advances in the development of accurate and labor-saving equipment to carry out the basic procedure. *It is important* that the procedures and techniques used in each survey be accurately described and documented so that later surveys can be carried out in a similar manner and provide comparable data.

6.3.4 Dating of Sediments

For reservoirs where no previous sediment surveys exist, there are various techniques for dating the existing sediment and thereby estimating the rate of accumulation.

6.3.5 Currents in Reservoirs

Wind shear and density differences generally dominate the forces which create water currents in lakes and reservoirs. Generally, the measurement of these currents create complex problems unless a heavy sediment-laden underflow exists. The measurement of sediment-laden flows is often important to assist in the operation of the reservoir for maximum sediment flushing.

6.4 METHODS OF PRESERVING RESERVOIR CAPACITY

The planning, design and implementation of measures to mitigate the undesired effects of reservoir sedimentation are important parts of a water resources project. They consist of measures to reduce the quantity of sediment entering the reservoir, to reduce the deposition of sediment in the reservoir and to recover part of the lost storage.

6.4.1 Techniques to Reduce the Entry of Sediment

In the long run, watershed management is the best method to reduce the yield of sediment and its entry into the reservoir. For large basins, subject to intensive erosion, such measures cannot give quick results and may not be implemented because of the high costs. The construction of auxiliary dams to retain sediment may have a quicker effect, but may not last long because of the filling of the small reservoirs with sediment. Bypassing sediment-laden flows during floods may be an effective method of diverting a large percentage of the sediment carried by the rivers before entering the reservoirs. Such techniques are appropriate in favorable topographical circumstances. All these possibilities should be considered and evaluated in the planning and design stages of the project. The construction of several reservoirs in cascades on the same river, in many cases, proves to be economical from the point of view of the useful life of the reservoirs on the downstream part of the river. Vegetative screens at the upstream end of reservoirs may withhold a significant part of the entering sediment.

6.4.2 Measures to Reduce Deposition of Sediment in Reservoirs

The deposition of coarse grains at the upstream end of the reservoir cannot be prevented. A large part of the finer particles can, however, be evacuated by the application of appropriate techniques.

Efficient methods are flood flushing and venting of turbid currents. The dam must be provided with adequate bottom outlets by which such operations can be efficiently executed. The bottom outlets should be straight, large enough to prevent choking by debris and safe against scour. It is essential to foresee the necessity for repairs in case they become damaged by the high velocity flow during flushing.

Venting of density currents is efficient in certain cases, depending upon the reservoir topography and sediment characteristics.

Both of the above methods require a certain quantity of water, which is lost for other purposes of the water resources project.

Prediction of the efficiency of flood flushing and density current venting must be made in the design phase and operational techniques, which optimize flushing or venting, should be adopted. During the operation of the project, measurements should be made to check the adequacy of the operating procedure and to verify the mathematical model used in the calculations. Operational techniques must be corrected, if necessary.

6.4.3 Recovery of Lost Storage

In many cases, lost storage can be partially recovered by removing parts of the deposited sediment from the reservoir, or at least, moved from the live or active part of the reservoir to the dead storage, below the level of the water intake structures.

The possible techniques vary depending upon local topography, availability of water for flushing, cost of storage capacity, etc. Storage recovery techniques should always be considered in the planning and design stages of the project, and provisions made for adequate structural features of the dam and appurtenances.

The most effective methods consist of operating the reservoir at low levels during floods, releasing flood discharges through bottom outlets rather than over spillways. Such practice is, of course, not possible if the flood water must be stored for later use.

Certain effects of sediment removal may be achieved by emptying and flushing operations, provided that water is available in the reservoir for that purpose. This is usually done after the irrigation season. The planning for such practices needs careful consideration of such things as the speed of drawdown, exist discharge, permissible sediment concentration in released discharges, etc.

Dredging of sediment deposits is a costly operation, which may be justified considering the economic value of water and the impossibility of replacement of lost reservoir capacity. Apart from the cost of dredging, the disposal of excavated mud may, in many cases, be a limiting factor.

In many cases, ecological considerations may limit, or even prohibit, flushing operations, because of damages inflicted to aquatic life downstream from the dam by the high sediment concentrations of the released water. Objections on ecological principles may, in some cases, be exaggerated and economically not justified. In some countries, the evacuated sediment may be used for improving agricultural land.

6.5 PREDICTION METHODS

Prediction of reservoir sedimentation and related phenomena are essential for decisions in all stages of a project: in planning, design, and operation of the reservoir and water resources system. The selection of prediction methods depends to a great extent on the objectives of the decisions for which the prediction is made.

6.5.1 Objectives of Prediction

A clear statement of the objectives must be prepared before the actual work on the prediction is undertaken. A list of possible objectives of the prediction is given in Table 5.1.

6.5.2 Uncertainties and Sensitivity Analysis

The prediction of reservoir sedimentation and related phenomena is based on calculations which contain many uncertainties, both in the data and in the modeling of the sedimentation process. It is, therefore, important to examine the influence of possible errors on the prediction and on the engineering decisions which will be based on them. The prediction thus obtained should be probabilistic rather than deterministic.

Financial and other constraints imposed on the sedimentation studies in most cases restrict the prediction to quasi-deterministic methods, based on averages and assumptions. It is recommended that this practice be changed and that sensitivity analysis be applied to all calculations used for the prediction.

6.5.3 Empirical Methods of Prediction

Empirical methods based on the trap efficiency concept may satisfy the requirements in preliminary stages of the project. They are limited to an estimate of the overall volume of sediments which may be trapped by the reservoir over a long period of time, and to a preliminary idea of the possible location of the deposits within the reservoir.

The methods still in use were developed years ago, and should be verified by a comparison with observation on a worldwide scale in order to be applicable in different climatic, topographical and hydraulic conditions.

6.5.4 Mathematical Modeling of Reservoir Sedimentation

Most models are based on one-dimensional calculations of water level and sediment balance, assuming steady nonuniform flow. It is strongly recommended that such models be used only in conjunction with field measurements.

Channel roughness should be calibrated by field measurements. The input of suspended sediment should be simulated on the ground of available records of sediment concentration. The grain size composition of the suspended sediments should be determined by field measurements. Since bed load in rivers cannot be measured with accuracy, empirical formulae must be used, the choice of which depends on experience and tradition. Empirical formulae for the transport of suspended sediments may be used after calibration by field measurements.

Since the sediment balance is calculated in terms of mass, and the channel modifications must be given in volume, the density of deposits is a very important parameter of the calculations. It must be determined, whenever possible, by field measurements, in the same reservoir or in analogous conditions. The model being one-dimensional, additional hypotheses must be introduced in order to calculate the modification of the channel cross sections. These assumptions should take local conditions into account.

The model can be calibrated only by comparing predicted and observed channel modifications, which is possible only after a few years of service of the reservoir. Prediction in the design stage, therefore, must contain an analysis of sensitivity to the variations of the relevant parameters within expected limits. The prediction thus obtained is necessarily probabilistic, particularly since the future input of water and sediment is probabilistic. The prediction should be improved after the implementation of the reservoir by repeated calculations and field measurements.

6.5.5 Evaluation of the Efficiency of Storage Recovery Practices

While the reservoir sedimentation can be represented by one-dimensional, steady flow models, storage recovery techniques such as emptying and flushing, retrograde erosion, and venting of density currents require more complex approaches.

Flushing of sediments by abruptly releasing the stored water from the reservoir is a prominently unsteady phenomenon, with rapid changes of the water level, flow velocities, sediment transport and channel topography. Although succussful models have been reported, no general recommendations can yet be made, except that any model applied should rely on field measurements and no model can be used without previous calibration. Real time prediction for the management of actual operations requires careful preparation and quick adaptation of the model on the spot to observations during the operation.

Retrograde erosion of sediment deposits by releasing high discharges through an empty reservoir requires complex modeling of unsteady two or three dimensional flow, with mobile boundaries.

Approximate calculation methods have been developed, nevertheless, based on semiempirical formulas, which need verification and adaptation to local conditions.

The prediction of the effects of venting of density currents is based partly on the theory of stratified flow and partly on empirical relationships whch are specific for turbidity currents.

Simple criteria exist for the determination of the point of plunge. The progression of the current can be approximated by one-dimensional equations of flow except near the dam, where two- or three-dimensional approximation is needed, depending upon the outflow conditions. Thanks to intensive research of stratified flow, improvement of the calculation methods is to be expected. Important questions, which are specific to suspended sediment transport by turbidity currents, need further research: the settling of sediment particles during the progression of the current, modification of the sediment concentration and the development of the front of the turbidity current are some of these questions. In addition to the theoretical and laboratory research, field measurements in reservoirs are of the greatest importance, due to the scarcity of available data.

6.5.6 Prediction of Erosion Downstream From Dams

Modeling of downstream erosion is similar to that of reservoir sedimentation: one-dimensional steady flow equations can be used for water level calculations and the sediment balance can be calculated by empirical sediment transport formulae. One of the main specific questions to be solved is the prediction of bed-armoring. Approximate semiempirical methods have been proposed, which take into account the grain size distribution of the bed material and the shear exerted by the flow. It would be very useful to collect and analyze the vast amount of data which exist today on bed deformation downstream from dams and to verify the methods in use.

6.6 RECOMMENDATIONS FOR FOLLOWUP ACTIVITIES

Based on the previously described conclusions and responding to the discussion and suggestions made at the Nanjing Symposium, the following activities are proposed within the 3rd Phase of the International Hydrological Programme:

1. To convene a workshop within the activity of the IHP-III and in conjunction with the interested NGO's under the general title:

"Advances in Reservoir Engineering"

with the purpose of discussing the following subjects:

a) Technical and socio-economic impacts of reservoir sedimentation and water quality deterioration:

b) Field measurement techniques

c) Methods of preserving the reservoir capacity and water quality

d) Prediction methods related to reservoir engineering problems

e) Management aspects of storage and water quality preservation

2. The preparation of a report on shallow lakes and surface water impoundments, covering the following points: wind induced and wave induced bank erosion and wind induced sediment movements (stirring-up and re-settling, creeping movements of soft bottom material) water quality aspects of intermittent sediment movement, overgrowing of aquatic vegetation; specific problems of downstream river reaches; additional bed and bank erosion caused by the peak energy production of low-head hydropower plants; erosion caused by modifying the natural flood hydrographs (cutting of the tails) etc.

3. The preparation of guidelines addressed to planners and decision makers on the sedimentation aspects of reservoir planning, design and operation, with special attention given to the structural elements of dams (e.g., bottom outlets), which are essential for the preservation of the storage capacity.

4. A call upon non-governmental organizations, and in particular upon ICOLD, IAHR, IASH to include the topic of reservoir sedimentation into the agenda of their forthcoming conferences. A world-wide review of the failure of reservoirs to perform their planned functions due to sedimentation would be a specially welcome contribution for the benefit of planners and engineers.

Appendix A

Measurement of Rates of Accumulation of Sediments From Radioisotope Data

by Bryon R. Payne

Carbon-14 dating of organic sediments has been used to estimate sedimentation rates during past millenia, but the method is not appropriate for relatively recent time which is of practical interest in problems of sedimentation in reservoirs. The presence of ^{137}Cs and ^{210}Pb in sediments provides two methods for estimating rates of sedimentation during the past 150 years. Caesium-137 ($T_{1/2}$ = 30y), which does not occur naturally, was produced in tests of nuclear weapons which commenced in the early 1950's. Therefore, the ^{137}Cs method is applicable to estimates of sedimentation rates from 1954 to the present time. Lead-210 is a naturally occurring radioisotope originating from the radioactive decay of uranium in the earth's crust. The half-life of ^{210}Pb (22.35y) and the accuracy with which measurements can be made enable the estimation of sedimentation rates up to about 150 years. Both methods require that the actual radiometric measurements be entrusted to a laboratory experienced in these measurement techniques.

The Caesium-137 method

Caesium-137 has been deposited on the earth as fallout. Its deposition has been measured at many locations in the world. Maximum deposition occurs in both hemispheres in the latitude bands 30°-60°. Deposition was at a maximum in 1963-64. Cumulative deposition of ^{137}Cs may be estimated from actual measurements of ^{137}Cs or inferred from ^{90}Sr data. In freshwater the ^{137}Cs is absorbed on the micaceous component of the sediment. During the accumulation of sediment, each layer will have a ^{137}Cs content determined by the deposition of ^{137}Cs at the time of formation of that layer. Since the fallout of ^{137}Cs has varied with time (Figure A.1), there will be a similar variation in the ^{137}Cs content of a sediment profile (Figure A.2). The ^{137}Cs profile in the sediment is obtained from core samples which should be taken so that negligible mixing losses occur. The cores are cut into 0.5, 1.0 or 2.0 cm sections. Samples (ca. 25-30g dry sediment) are measured in a calibrated low background Ge(Li) detection system.

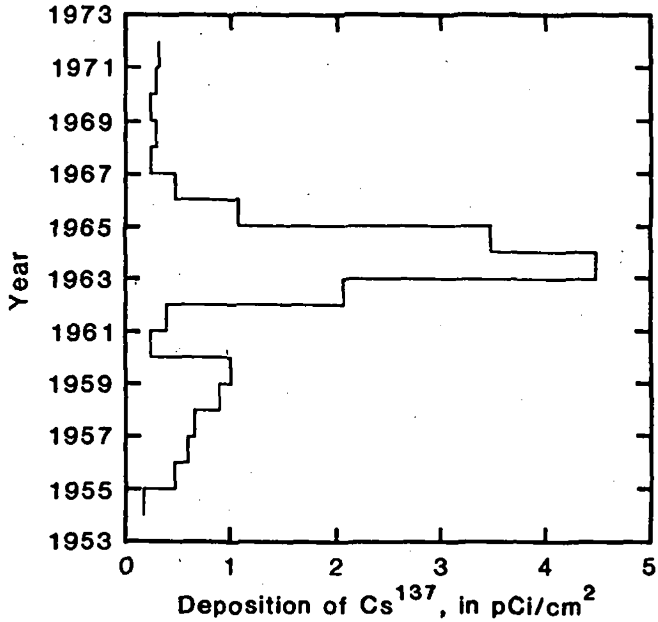

Figure A.1. An example of the annual deposition of ^{137}Cs

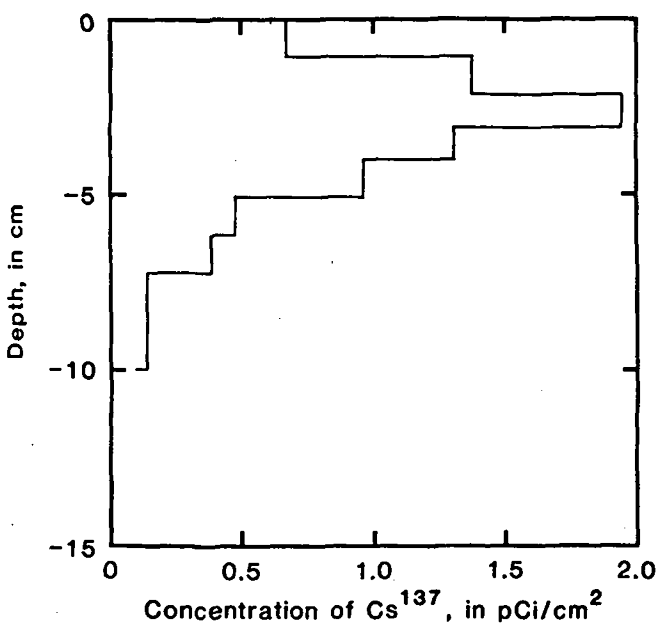

Figure A.2. A hypothetical distribution of ^{137}Cs in lake sediments

A profile of ^{137}Cs concentration against depth (Figure A.2) shows a maximum activity at a depth which corresponds to 1963 and the depth where ^{137}Cs is first detected which corresponds to 1954. Thus, the sedimentation rate may be estimated for the period 1963 to present (R_1) and also for the period 1954 to present (R_2).

$$R_1 = \frac{D_{max}}{(Y-1963)}$$

$$R_2 = \frac{D_o}{(Y-1954)}$$

where D_{max} = depth of maximum ^{137}Cs concentration corresponding to 1963

D_o = depth of first appearance of ^{137}Cs in core corresponding to 1954

Y = year of core sampling.

The Lead-210 method

The radioactive decay of uranium in the earth's crust produces a series of daughter products. One of these is Radon-222 ($T_{1/2}$ = 3.8d) which is released as a gas from the earth's crust and surface waters to the atmosphere. The ^{222}Rn decays through a number of short-lived radioisotopes to ^{210}Pb. The ^{210}Pb rapidly becomes attached to aerosol particles which reside in the atmosphere for at the most a few weeks, depending upon latitude, season, frequency of precipitation, size, and altitude of aerosols. Thus, there is a continuous flux of ^{210}Pb to the earth's surface. This flux is termed unsupported ^{210}Pb since it is not in secular equilibrium with its parent ^{222}Rn as in the case of ^{210}Pb. The incorporation of this unsupported ^{210}Pb in lake sediments results in an excess of activity which is measured in a sediment core.

As in the ^{137}Cs method, cores are cut into sections, but the sample processing prior to measurement of ^{210}Pb is more laborious. A typical procedure is to add a known amount of ^{208}Po to a 3-5g aliquot of a core section to determine the yield of the radiochemical separation leading to the preparation of a source of ^{210}Pb - for alpha counting in a low-background, high-resolution, silicon-surface barrier detection system.

The concentration of unsupported ^{210}Pb will decrease as a function of depth of its radioactive decay. Provided there is no significant migration within the sediment, the ^{210}Pb activity will decrease exponentially with depth (Figure A.3). The age of the sediment at any particular depth can then be estimated from the unsupported ^{210}Pb activity at that depth relative to that at the water-sediment interface.

$$A_x = A_o e^{-\lambda t}$$

$$\text{or} \quad t = (1/\lambda) \ln (A_o/A_x)$$

where t = age of sediment at depth x
A_o = activity of unsupported ^{210}Pb at water-sediment interface
A_x = activity of unsupported ^{210}Pb at depth x
λ = radioactive decay constant for ^{210}Pb

The ^{210}Pb activity may not always exhibit an exponential decrease with depth. This may be due to one or more reasons. For example, there may be disturbance or incomplete recovery of the upper layers of sediment by the coring device. There may be even actual changes in rates of sedimentation. Barnes et al. (1979) have reported on the dating of sediments from Lake Washington. Three different sedimentation rates were estimated. Data for the deepest sediment profile indicated a rate of 0.063 cm y^{-1} prior to 1889. A decade or so after this time, the rate was estimated to be 0.83 cm y^{-1}. More recent sediments were concluded to have been deposited at more than four times the rate of the deeper sediments. The authors interpret these differences in sedimentation rates in terms of the changes in land use and other environmental factors.

Robbins and Edgington (1975) have estimated the sedimentation rates in Lake Michigan on the basis of cores from eight locations which indicate that rates over the past hundred years or so have been constant. The method was found to be in good agreement with an estimate based upon the distribution of pollen. These authors also studied the distribution of ^{137}Cs in the cores and suggest that the use of both methods is to be recommended, since the combined approach can provide a better insight to the physical and chemical nature of the sedimentation process.

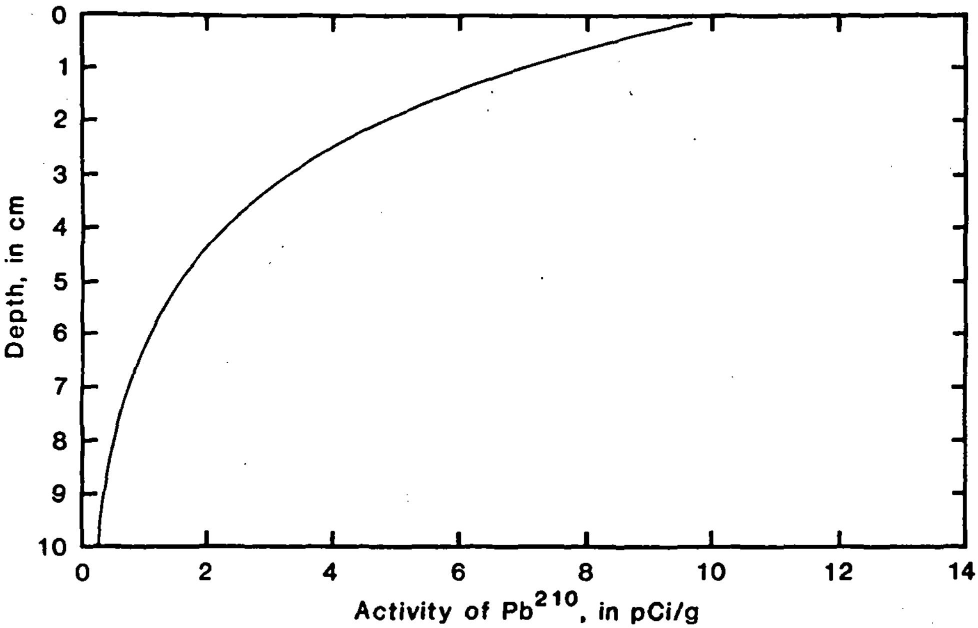

Figure A.3. A hypothetical profile of ^{210}Pb activity with depth in lake sediments

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