River Response to Hydraulic Structures

by S. Raynov
D. Pechinov
Z. Kopaliany
Edited by R.D. Hey

Technical Documents in Hydrology

International
Hydrological Programme

United Nations Educational,
Scientific and Cultural Organization

Unesco
Paris, 1986

Technical Documents In Hydrology

RIVER RESPONSE TO HYDRAULIC STRUCTURES

by the Editorial Group
on IHP-II Project A.3.8

S. Raynov (Bulgaria) Chairman
D. Pechinov (Bulgaria)
Z. Kopaliany (USSR)

Edited by R.D. Hey (U.K.)

UNESCO, Paris, 1986

SC.86/WS/48

PREFACE

Although the total amount of water on Earth is generally assumed to have remained virtually constant during recorded history, periods of flood and drought have challenged the intellect of man to have the capacity to control the water resources available to him. Currently, 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 resource. 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 worldwide 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 the relevant national activities and to participate in regional and international cooperation within the IHD programme. The knowledge of the world's water resources 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 multi-purpose 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 publication series are issued: "Studies and Reports in Hydrology" and "Technical Papers in Hydrology". In addition to these publications, and in order to expedite exchange of information, some works are issued in the form of Technical Documents.

CONTENTS

FOREWORD

One of the objectives of the second phase of the International Hydrological Programme (1981-1983) was to 'stimulate and support studies concerning the determination and prediction of the influence of man's activities on the hydrological regime and of the interactions of these activities with the environment'. Under this programme objective the IHP-II Project A.3.8 was undertaken to study methods of assessing the 'influence of hydraulic structures on sedimentation processes and flow regime'.

The Intergovernmental Council of the IHP approved the proposal of the Bulgarian National Committee for the IHP to take the responsibility for the execution of this project.

The Bulgarian National Committee for the IHP established a Working Group under the chairmanship of Professor S. Raynov. In April 1984, the Bulgarian National Committee organized, with the support of Unesco, a Workshop on 'Evaluation of the Influence of Hydraulic Structures on River Bed Processes Downstream from Dams' in which experts from Argentina, Bulgaria, Cuba, Czechoslovakia, Egypt, Federal Republic of Germany, Poland and the USSR participated.

Two meetings of the Editorial Board took place in Paris (August 1985, June 1986) consisting of Prof. S. Raynov (Higher Institute of Architecture and Civil Engineering, Bulgaria), Dr. D. Pechinov (Institute of Hydrology and Meteorology of the Bulgarian Academy of Sciences, Bulgaria), Dr. Z. Kopaliany (State Hydrological Institute, USSR) and Dr. R.D. Hey (School of Environmental Sciences, University of East Anglia, U.K.). Dr. M. Rusinov, IHP Secretariat, provided the technical secretariat.

Unesco is much indebted to the following experts who assisted in the preparation of this report: R.A. Lopardo (Argentina), E.V. Davis (Cuba), V. Strauss (Czechoslovakia), S. Shalash (Egypt), S.N. Boulos (Egypt), F.H. Weiss (FRG), Z. Mikulski (Poland).

1. INTRODUCTION

Rivers, under natural conditions, are continually eroding or depositing. This can be considered as either reversible or irreversible.

Reversible changes result from the natural cycle of water and sediment run-off from a catchment. The river adjusting its shape and dimensions in response to any inbalance between sediment supply and its transport capacity. When the supply of sediment is less than the transport capacity of the river, erosion occurs. This eventually ceases when the increased cross-sectional area, reduced channel slope and velocity decreases the sediment transport capacity of the river. When the input load exceeds the transport capacity, deposition occurs. This will cease when the decreased cross sectional area, increased slope and velocity, increases its transport capacity.

Irreversible changes normally occur under natural conditions in the upper and lower reaches of a river. In the upper reaches systematic erosion occurs, while deposition results in the lower reaches. In natural rivers these changes take place so slowly that for engineering and economic assessment purposes they may be ignored.

Man-made activities, for example, change in land-use, construction of dams, interbasin water transfers and regional river regulation schemes, river stabilisation and training activities, flood control, dredging and straightening can all lead to significant changes in the discharge regime and volume and calibre of the sediment load, which can result in systematic erosion or deposition (table 1.1). Any increase in transport capacity or decrease in the quantity or calibre of the sediment transported will precipitate erosion.

Decrease in the transport capacity or any increase in the volume or calibre of the sediment being transported will cause deposition. The intensity of these erosional or depositional phases depends on the degree of inbalance between sediment supply and transport capacity and, in the case of erosion, on the calibre of the bed material.

Any man-induced change which alters the hydraulic characteristics or hydrological regime of a river can cause erosion or deposition. There are numerous examples in which sudden, detrimental and very often catastrophic changes have occurred, which affect both engineering structures and the river environment, as a result of ignoring the flow processes operating in alluvial channels. The primary aim of this report, implemented under IHP II project A.3.8, is to present case-studies of river response to the construction of engineering structures and the implementation of river basin management and development programmes, in order to identify the degree and nature of erosion and deposition so that they may be taken into account when designing new structures and protecting existing ones. In addition empirical and numerical modelling procedures are outlined for predicting erosion and deposition and changes in channel form as a result of the construction and operation of river engineering works and dredging activity.

Table 1.1 Causes of channel instability

Characteristic	Nature of change	Type of channel instability	Cause of instability
Transport capacity	increase	degradation	1. Canalization and training works 2. Downstream of river outfall 3. Reduction in base level 4. Upstream of dredged reach
	decrease	aggradation	1. Upstream from reservoirs 2. Downstream from river intake 3. Rise in base level 4. Downstream end of dredged reach
Sediment discharge	increase	aggradation	1. Increase of surface erosion from drainage basin 2. Increase in sediment production as a result of building works
	decrease	degradation	1. Downstream from reservoirs 2. Downstream from dredged reach 3. Decrease of surface erosion from drainage basin

2. CHANNEL INSTABILITY DUE TO RESERVOIR CONSTRUCTION

Prior to 1973 there were 1347 reservoirs in the world with capacities greater than 100 $\times 10^6$ m^3 (total capacity of 4100 $\times$ 10^9 m^3). In addition there were between 10 and 20,000 smaller reservoirs (total capacity about 200 $\times 10^9$ m^3). Their combined capacity, 4300 $\times 10^9$ m^3 is 6% of the mean annual run-off of the world's rivers (Lvovitch, 1974). Given the increased need for water for power, navigation, agricultural use, and human consumption and the disparity of areas of water supply and areas of demand, reservoir construction will continue at an accelerated rate.

One of the most important problems relating to the construction and operation of reservoirs is that of erosion and deposition;

sedimentation in the reservoir and erosion downstream from the dam. Consider a reach of alluvial channel that was stable prior to dam construction. The reservoir becomes a trap for the sediment run off from upstream, while downstream from the dam irreversible erosion occurs due to the fact that transported material is not replaced by sediment from upstream.

When designing a new dam, it is necessary to predict the rate of sedimentation in the reservoir, as this influences its operational efficiency and design life. Once constructed, especially if accretion is rapid, systematic measurements of hydrological and morphological parameters should be taken in order to provide data to aid the improvement of existing mathematical models or aid the development of new ones.

Erosion downstream from dams and hydro-electric power stations is often not considered in detail with the result that problems can occur due to bed scour (reconstruction of engineering structures), reduction in water levels (additional controls to maintain navigation depths) and decrease of groundwater levels. Erosion is controlled by a number of factors - type and period of construction, regulating capability of the reservoir, operational regime of hydro-electric power stations, sediment run-off regime and type of river bed structures, rip-rap calibre, change in run-off and sediment yield from tributaries in response to reservoir sedimentation and downstream erosion. Predicting the response of a particular river to these changes, both qualitatively and quantitatively (rate and degree of change), is still relatively difficult.

2.1. RESERVOIR SEDIMENTATION

Due to the decrease in the velocity of flow in the reservoir, bed load and some suspended load is deposited. The amount depends on the regulating capability of the reservoir and the calibre and volume of the sediment load.

For example, in the first 10 years following construction of Iron Gate I dam on the Danube, all the bed load was trapped. Suspended sediment load coarser than 0.2mm was deposited, although proportions of the suspended load finer than 0.2mm were transmitted downstream (table 2.1).

The trap efficiency of the reservoir, which can be expressed by the relation $V_{s\ dep.}/V_s$, where $V_{s\ dep.}$ is the amount of sediment deposited per year and V_s is the input sediment load per year, depends on its regulating capability, characterized by the relation V_r/V_Q, where V_r is the reservoir capacity and V_Q the annual inflow to the reservoir (fig. 2.1).

Table 2.1 Suspended sediment grain-size distributions: input load, deposited material, transmitted load. Iron Gate I Reservoir, River Danube (Raynov et al., 1977).

Grain diameter (mm)	Input load %	Deposited material		Transmitted load	
		relative to total input load %	relative to input grain size %	relative to total input load %	relative to input grain size %
above 0.2	12.1	12.1	100	0	0
0.2-0.1	12.5	12.3	98	0.2	2
0.1-0.06	9.3	8.8	95	0.5	5
0.06-0.02	29.0	26.7	92	2.3	8
0.02-0.01	15.1	11.1	74	4.0	26
below 0.01	22.0	15.0	68	7.0	32
TOTAL	100.0	86.0		14.0	

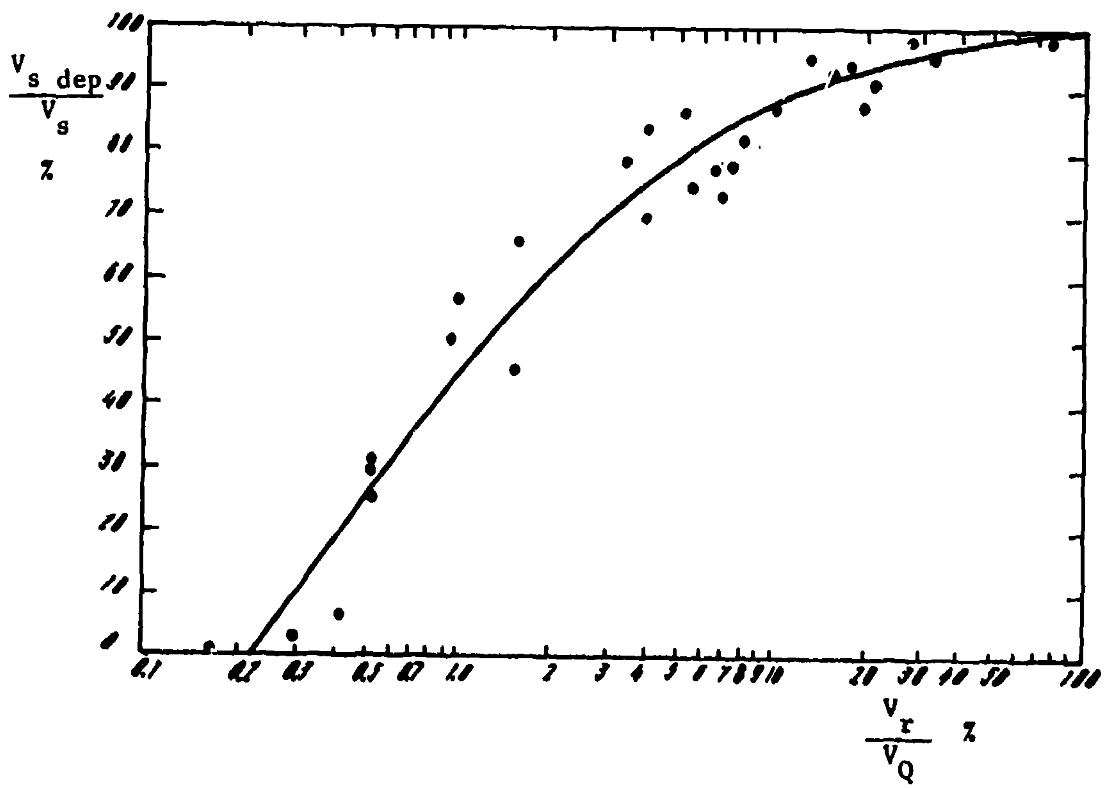


Fig. 2.1. Relation between reservoir trap efficiency $V_{s\,dep.}/V_s$ and the regulating capability V_r/V_Q (after Brune, 1953)

This indicates a direct relation between regulating capability and trap efficiency. However, even for reservoirs with low regulating capabilities relatively high trap efficiencies pertain. Thus for a regulating capability of 1%, 45% of the input sediment load remains in the reservoir, for 10% - 85% is deposited, and for 100% - almost all the input sediment is retained. Immediately post construction, a reservoir has maximum trap efficiency but this decreases with time as a result of its regulating capability being reduced by sedimentation. According to Shalash (1982), the sediment discharged through the High Aswan Dam in the first years post dam construction was 1-2% of the total sediment yield but this will gradually increase to approximately 8% after 500 years when it is expected that its dead storage capacity will be filled (Fig. 2.2).

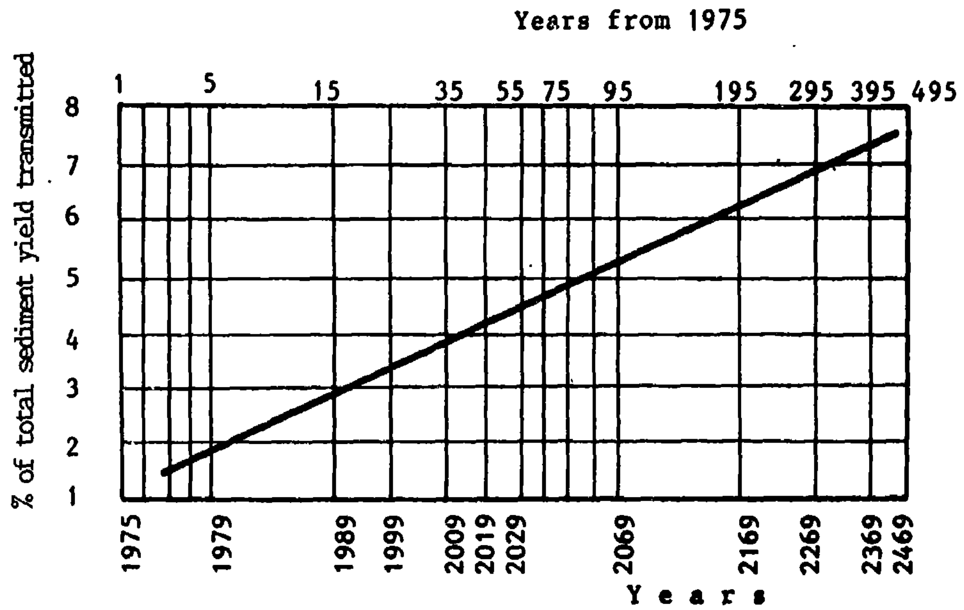


Fig. 2.2. Projected changes in the sediment discharge below High Aswan Dam (after Shalash, 1982)

Karasev (1975) suggested that there were 3 stages in the process of reservoir sedimentation:

1. when all, or practically all, the input sediment was deposited;
2. when sedimentation in the reservoir influences the volume of sediment transported past the dam;
3. when all input sediment was transmitted past the dam.

Skrylnikov (1961) observed that for several reservoirs the transition from the first stage to the second occurred when the reservoir volume (V_r) is given by the following empirical equation:

$$V_r = 8.3\ V_b$$

where V_b is the original bankfull volume of the river channel that has been drowned out by the reservoir. In the second, and especially in the third phase, downstream erosion decreases or ceases and deposition may occur re-establishing the pre-dam condition. The river Ihn downstream from the Etenbach reservoir (Fig. 2.3), is illustrative of this process.

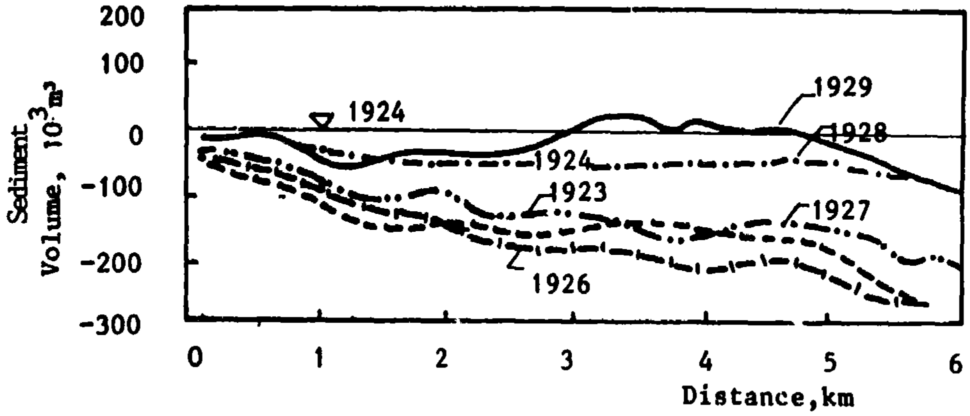


Fig. 2.3 Volume of sediment eroded or deposited downstream from Etenbach reservoir on the river Ihn.

The type of reservoir (lake, river), water level regime and grain size characteristics of the input sediment load control sedimentation patterns. The coarse fraction is deposited first, while fine material is deposited nearer the dam (fig. 2.4). The water level regime in the reservoir plays an important role in the distribution of sediment and their movement towards the dam. This fact is often used for reservoir flushing, enabling deposited material to be discharged downstream, which can lead to decreased erosion below the dam.

As was emphasised above, sediment size strongly influences the amount of material that is deposited or transmitted through the reservoir. Coarse material leads to increased deposition and a reduction in the amount transmitted and vice versa with fine material (table 2.1).

This fact should be borne in mind when predicting rates of reservoir sedimentation since this influences the degree of bed degradation downstream.

Field studies of reservoir sedimentation have been carried out since the end of the last century by Schoklitsch (1914), Poliakov (1935), Orth (1934), Levi (1938), Shamov (1939), Brune (1953), Altunin (1958), Wetter (1953), Mostkov (1950), Lisitzina (1969), Rossinsky and Kuzmin (1964), Karaushev (1965, 1966), etc. On the basis of field observations and sediment transport theories, it is possible to calculate rates of reservoir sedimentation. For detailed assessment of these methods reference should be made to "Instructions for computation of reservoir sedimentation at design stage" (1973) and "Sediment control methods" (1973) and publications of the IHP Programme Project A.2.6 "Methods of computing sedimentation in lakes and reservoirs" (1985) and Project 5.3 "Sedimentation problems in river basins" (1982). Consequently there is no need to consider in detail the existing methods for computing reservoir sedimentation, other than emphasizing the need to take account of the influence of reservoirs on the flow regime and sediment yield downstream from dams.

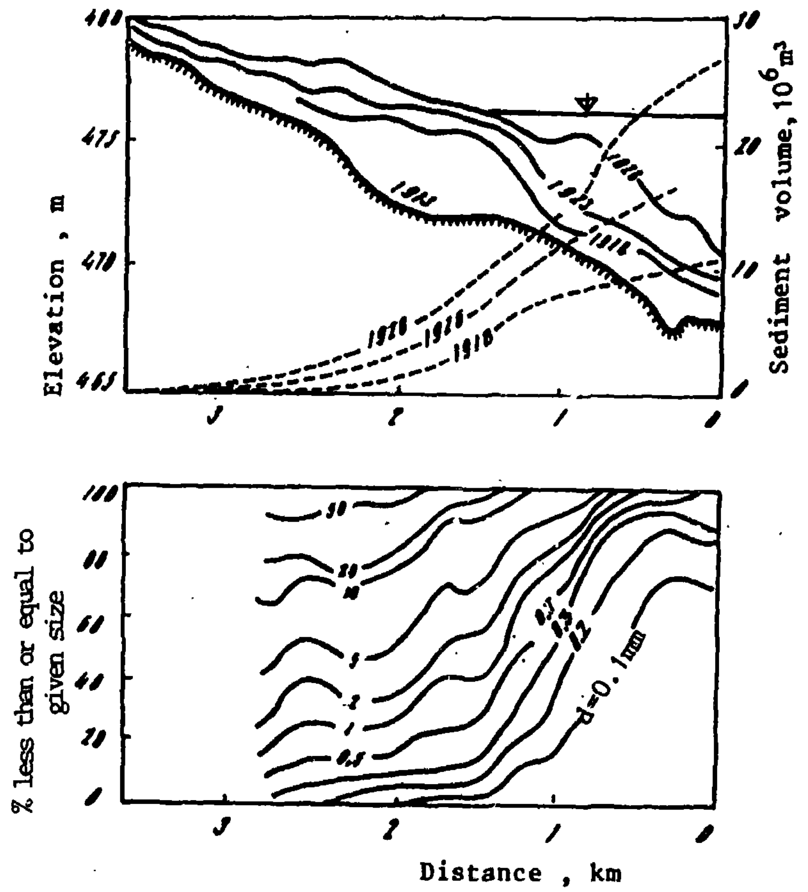


Fig. 2.4 Longitudinal profile, sediment volume and sediment grain size in the Saalah reservoir

2.2 CHANNEL INSTABILITY DOWNSTREAM FROM RESERVOIRS

Changes in the flow and sediment transport regime of the river due to the construction and operation of a reservoir can be responsible for channel instability downstream from dams. Redistribution of river run-off and a decrease in sediment yield due to deposition of coarse sediment upstream in the reservoir are observed in the post-dam period. Released water with a transport capacity exceeding the sediment supply causes river bed erosion. Irreversible erosion continues until a new equilibrium between the transport capacity of the flow and sediment supply is achieved. Due to erosion, and a related increase in channel cross sectional area and bed material size and a decrease in river slope, the transport capacity gradually decreases, while the volume and calibre of sediment released from the reservoir may eventually increase as a result of a reduction in its capacity to store sediment (Fig. 2.2). Eventually erosion ceases when increased sediment supply equals the reduced transport capacity. Rivers with bedrock outcrops, or those that armour, due to the preferential removal of fine material, will quickly stabilize.

A further increase in the sediment supply may cause deposition downstream from the dam (Fig. 2.3). For reservoirs with a small regulating capability, reduction in sediment yield may only be temporary. In contrast for those with a large regulating capability, such a reduction could be maintained for hundreds of years.

During dam construction and in the immediate post-dam period additional instability may result just downstream from the dam due to local flow turbulence or changes in flow direction due to the design and operation of the structure.

Prediction of river bed instability downstream from hydro-electric power stations is of great importance with regard to establishing optimum operation conditions for the station. During the design of the station, prediction of possible changes in river bed and water level elevations are necessary in order to 1) ensure its structural stability, 2) determine seepage losses, the location of turbines and the height of locks 3) provide the required depths for navigation, 4) assess the stability of engineering works downstream (bridges, supporting and quay walls, pipe-lines under the river bed, pumping stations, water intakes), and 5) evaluate the influence of reduced river levels on ground water conditions and, as a result, on the vegetation of the adjacent sections of flood plain. The effect of reservoirs on the flow and sediment transport regime of the river and their influence on bed and water levels, slope, width, sediment transport and calibre of bed load, channel form and river bed dynamics is discussed in the following section.

2.2.1 Influence of reservoirs on the flow regime of a river

The natural flow regime of a river is characterised by its variability both within and between years. Thus, during periods of high run off, i.e. 1-2 months per year, over half the annual flow can occur, while in the rest of the year water discharges are relatively low. This natural flow variability causes problems for resource development purposes. In low flow periods, navigation depth and generated output from hydro-electric power stations decrease, while water needs for irrigation increase. As a result, it is necessary to regulate flows by storing water in reservoirs during periods of excess flow in order to supplement periods when flow is deficient. River regulation is now universally accepted as a means of rationally developing regional water resources. Various periods of run-off control are possible, long term, annual and diurnal, depending on the run-off regime and the technical-economic considerations relating to the construction of reservoir with a given capacity. For long term regulation water can be stored from one year to another, while for annual (seasonal) regulation the redistribution of streamflow, even in dry years, occurs within the year. Diurnal flow regulation is carried out over a day and night. This also covers the case when a comparatively uniform streamflow is modified into a non-uniform one to meet requirements for the generation of electricity. Diurnal regulation requires comparatively small reservoirs, while long term regulation necessitates large reservoirs. Reservoirs with potential for long term regulation may also allow annual and diurnal regulation, while those capable of annual regulation may enable diurnal regulation.

The ratio of reservoir capacity (V_r) to annual flow capacity (V_Q) in the reservoir is a useful measure of streamflow regularity. The larger the value of this ratio, the greater the ability of the reservoir to regulate streamflow. For example, the relation (V_r/V_Q) for the Rybinski reservoir on the river Volga, with storage capacity of 1.67×10^9 m^3 and mean annual water discharge of $1120 m^3/s$, is 47% (figure 2.5) and the complete control of streamflow is possible.

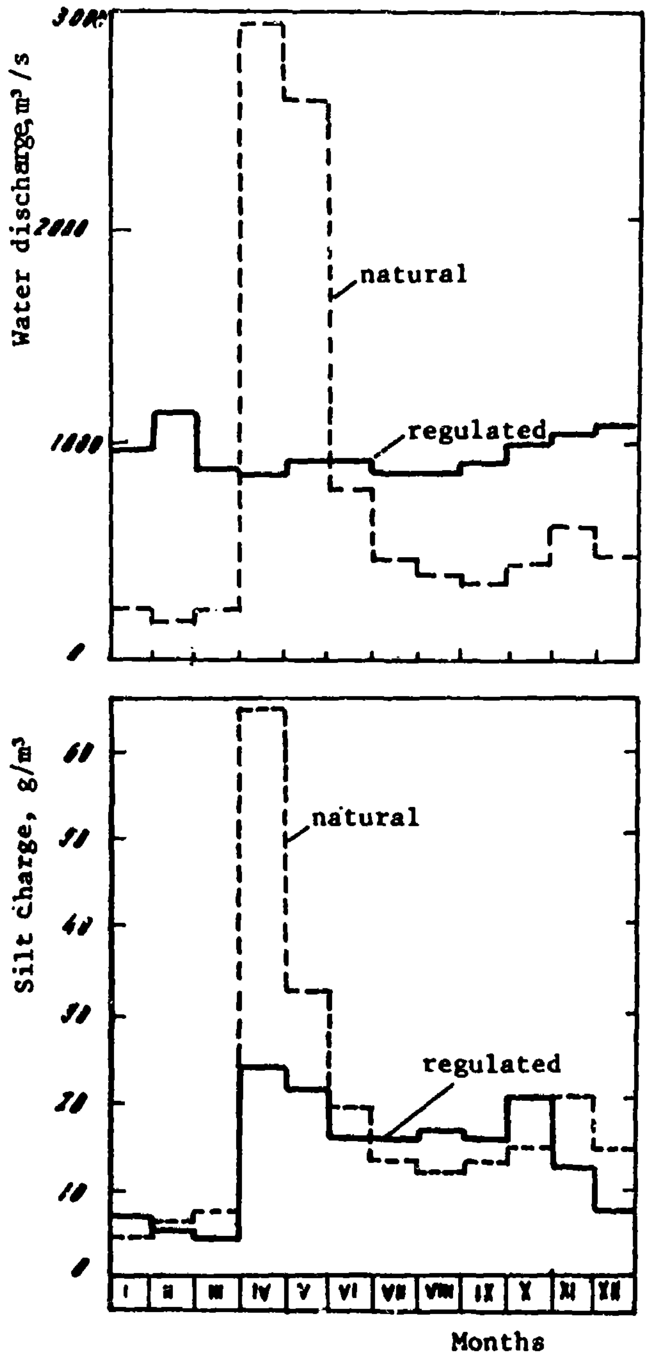


Fig. 2.5 Annual variation in monthly water discharges and silt charges on the river Volga downstream from Rybinsky reservoir (after Shamov, 1954)

For small values of the ratio V_r/V_Q, streamflow is relatively unaffected. For example, the relation V_r/V_Q for the Iron Gate I reservoir on the Danube, with a capacity 2.5×10^9 m^3 and mean annual water discharge 5700 m^3/s, is 15%, and little change in the annual distribution of streamflow is observed (figure. 2.6).

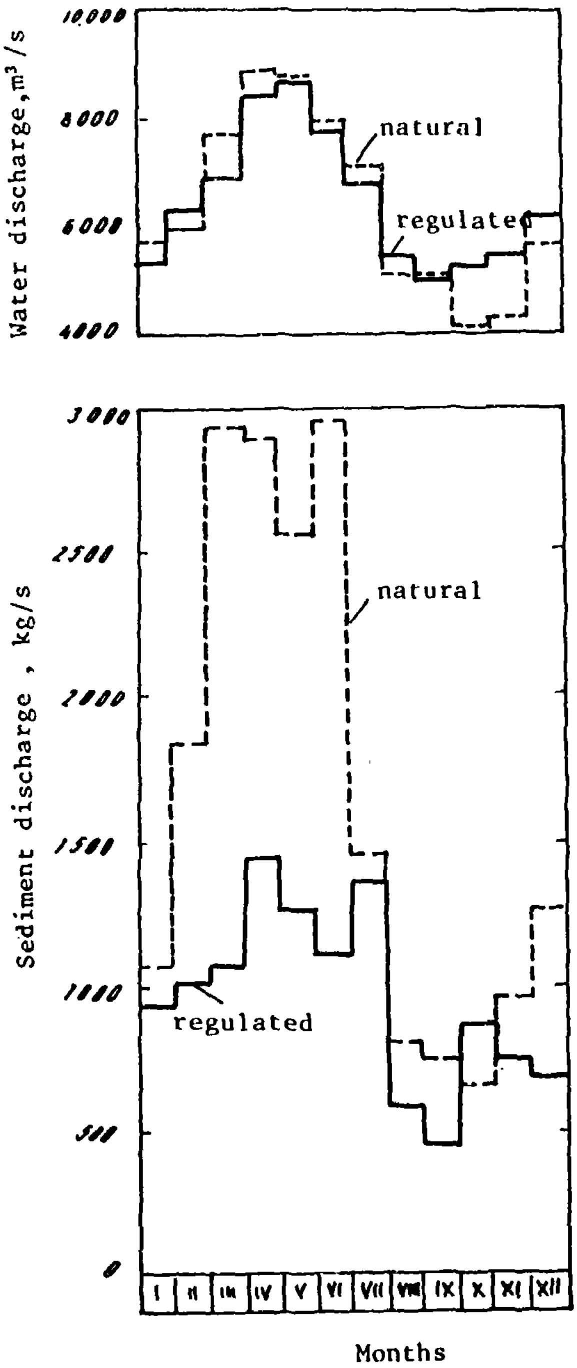


Fig. 2.6 Annual variation in monthly water and sediment discharges on the river Danube at Svishtov before and after closure of Iron Gate I dam (after Raynov et al., 1977)

Reservoirs can considerably modify run-off downstream from a dam. They store water during floods and periods of excess run-off. Floods passed through the reservoir will be attenuated and in some cases, the total volume of the flood run-off may be completely stored;

Stored water is released during dry periods, depending on the use of reservoir, for irrigation, water supply, power generation or navigation purposes.

An absolute decrease of river run-off can occur downstream from reservoirs used for irrigation and water supply. In many cases water is diverted directly from the reservoir through pipelines or canals with the consequence that small or zero releases are often made downstream. In contrast for some hydro-electric power stations, all or a part of the flow is diverted from the river and is returned at some considerable distance downstream. For example at Farhadsky hydro-electric power station on the river Syr Daria, diverted water is returned to the river 21km further downstream. In such cases releases are relatively unaffected by the control capability of the reservoir.

However, in general, releases reduce the frequency of flood discharges which means that the transport capacity of the river is reduced. This is illustrated in figure 2.7 where the annual discharge hydrograph is less peaked after dam closure. As a consequence the computed sediment transport rates post-dam construction decrease, declining from a mean annual value of 12kg s^{-1} to 9kg s^{-1}, assuming the bed material size does not vary. If the bed material size downstream from the dam is doubled by erosion, then the average transport rate is reduced to 7kg s^{-1}. This indicates that sediment transport rates downstream from the dam never achieve pre-construction values.

With hydro-electric power stations, power requirements often result in diurnal and weekly flow patterns (fig. 2.8) which can have a significant influence for considerable distances downstream. For example, the amplitude of the diurnal variation in water stage below Rybinsky dam on the Volga is as much as 3.5m and its influence is observed for up to 150km; downstream from the Iron Gate I dam on the Danube these figures are 3m and 250km respectively; on the Ob the amplitude is 2m at Novosibirsky dam reducing to 20cm 90km downstream. Diurnal variations in discharge at Kuibiskevsky hydro-electric power station are in the range of 0-13000 m^3/s. Water levels vary by a maximum of 4m, and stage changes are observed for over 200km below the dam. Any increase in discharge variability intensifies erosion processes downstream from the dam and is especially severe immediately downstream before any attenuation can occur.

2.2.2 Influence of reservoirs on sediment run-off

Reservoirs are effective sediment traps. Bed load and a considerable part of the suspended load are usually trapped even in cases where regulating capability is comparatively low. Although releases are deficient in sediment, the transport capacity of the river is decreased. The net result is bed erosion over a considerble distance downstream from the reservoir.

Williams and Wolman (1984) present data to show the effect of the Hoover dam on suspended sediment loads in the Colorado river. Comparisons of annual sediment discharges were made for two sites, one located 430km upstream from the reservoir and the other 180km downstream from it (Fig. 2.9). Prior to dam closure in 1936, the annual sediment discharge at both sites were comparable irrespective of the temporal variability of suspended sediment run-off. After dam closure, sediment discharge at the site upstream from the dam remained large, with considerable variation from year to year, while at the downstream site sediment discharge was considerably decreased as a result of sediment being trapped by the dam. It should be noted that during the period of dam operation sediment run-off continuously decreased downstream from the dam, probably due to the gradual removal of fine bed material in the course of bed erosion.

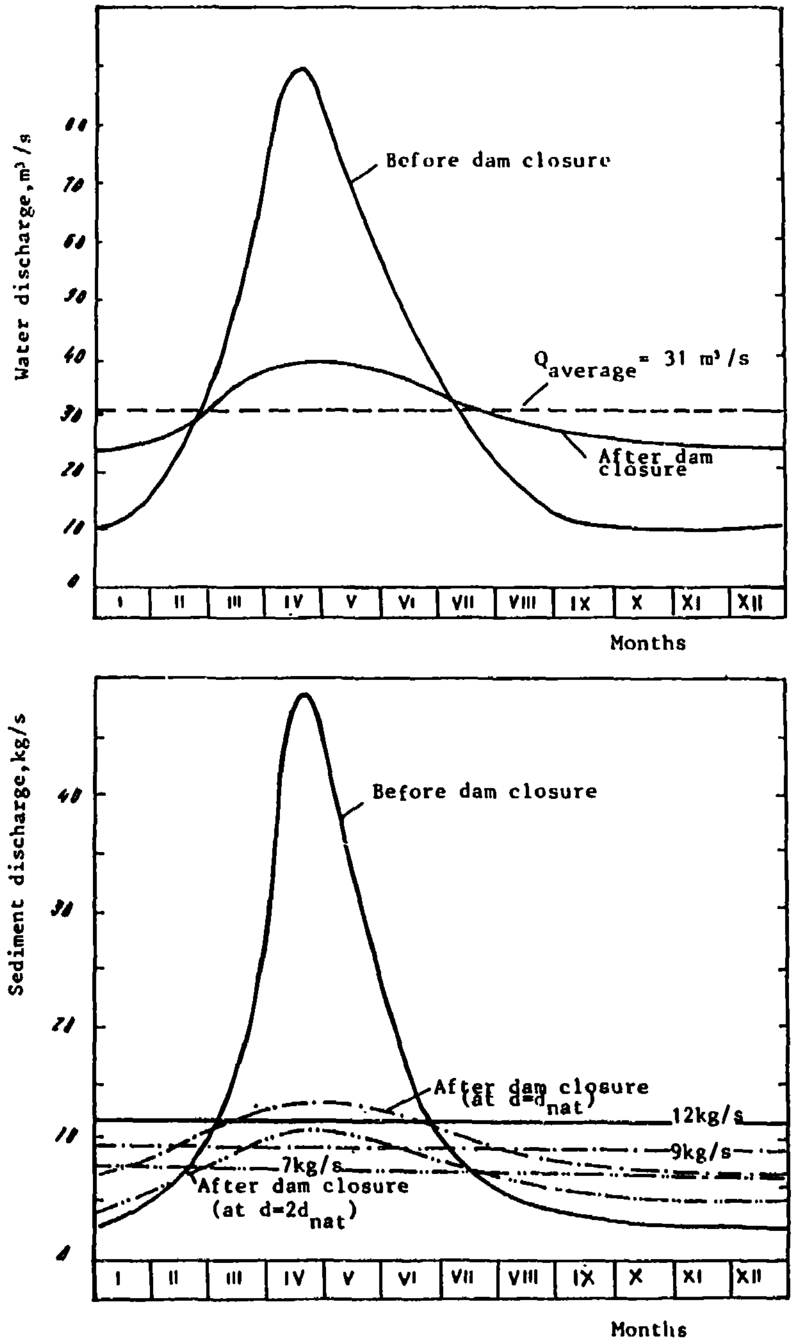


Fig. 2.7 Typical example of water and sediment discharges downstream from a dam before and after dam closure (d = bed material size)

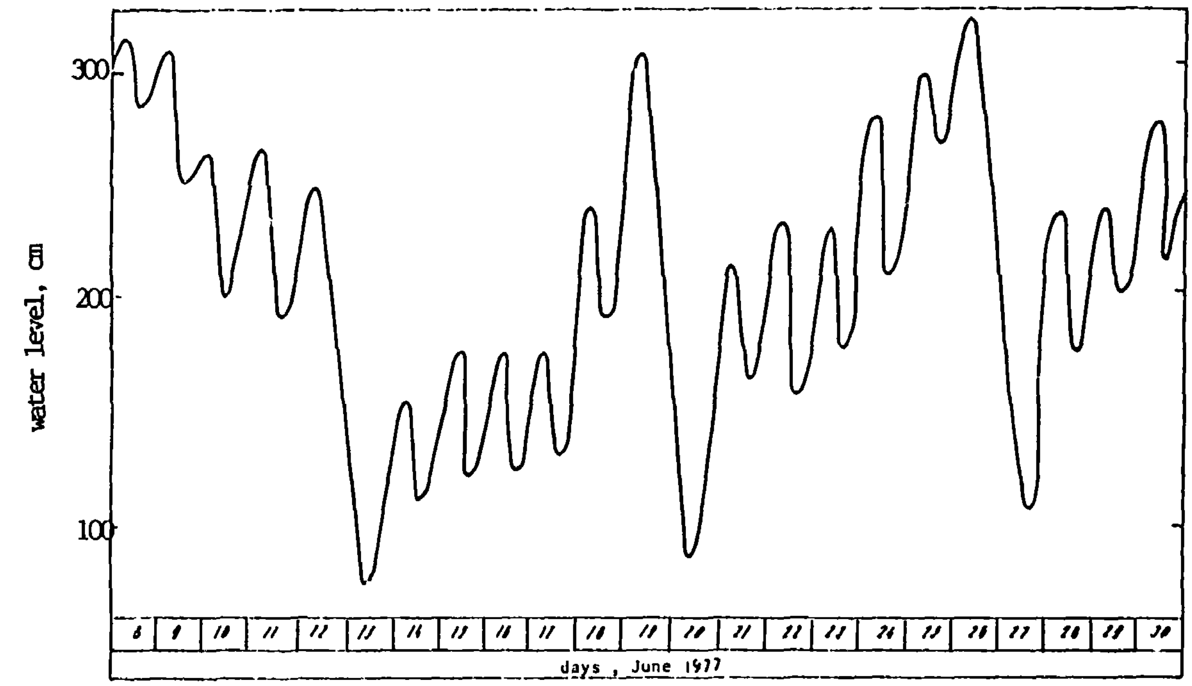


Fig. 2.8 Variability of water levels on the river Danube at Novo Selo 109km downstream Iron Gate I dam

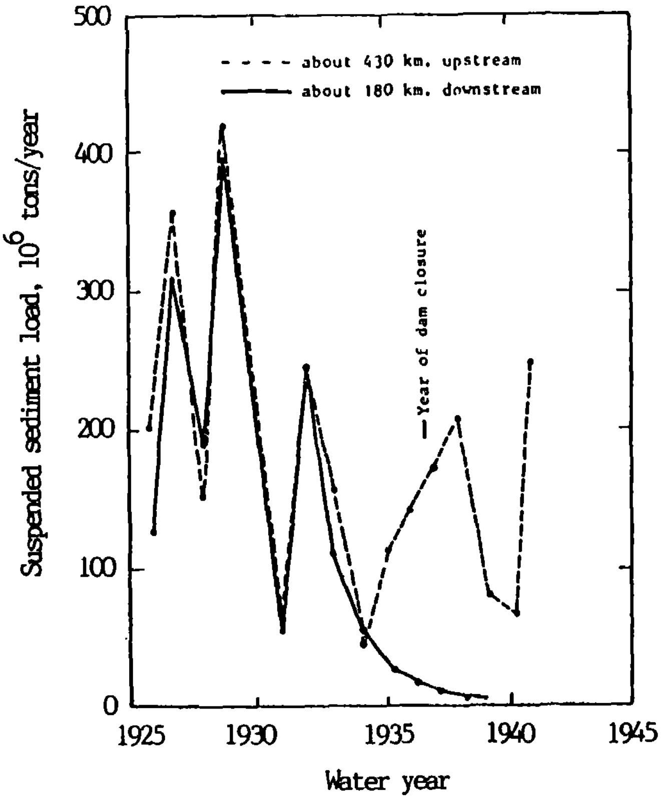


Fig. 2.9 Variation in annual suspended sediment load before and after closure of Hoover Dam, river Colorado, at a station 430km upstream from the dam and 180km downstream from the dam (after Williams and Wolman, 1984)

Suspended sediment concentrations can also be significantly affected by dam construction. Williams and Wolman (1984) present data for concentrations on the North Canadian river (Figure 2.10) which indicate that at a site located 45km upstream from Canton reservoir they did not change over time while 5km downstream from the dam there was a massive decrease in concentrations. Even 140km downstream from the reservoir concentrations were considerably decreased. Although bed erosion and tributary contributions increase sediment concentrations at a distance of 180km downstream, they are still lower than they were prior to dam construction.

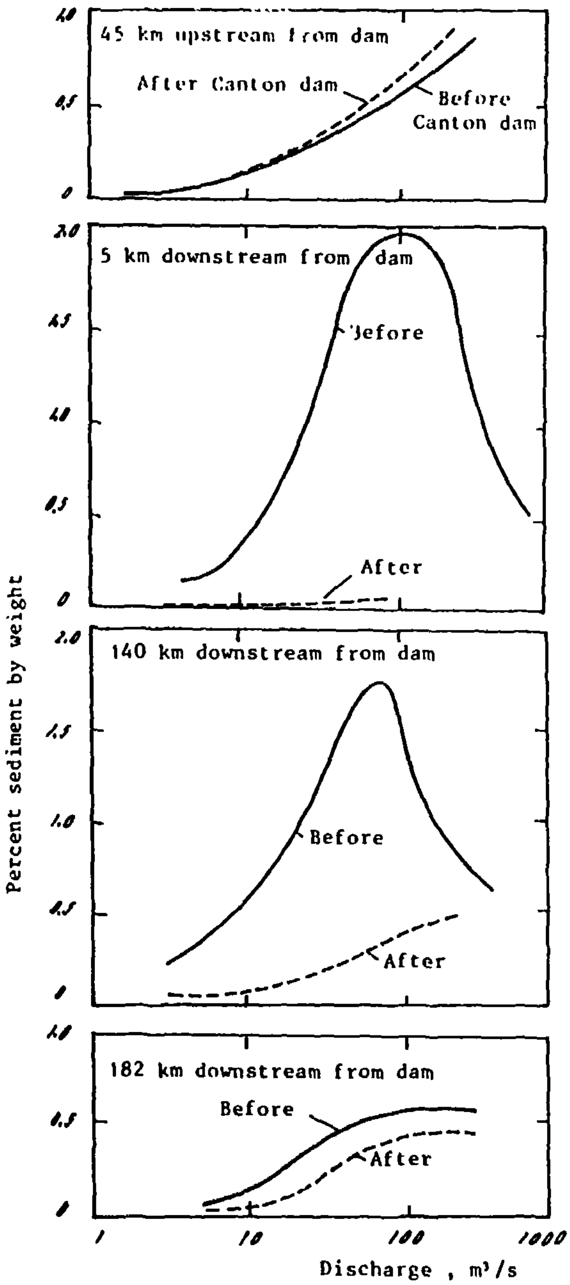


Fig. 2.10 Suspended sediment concentration discharge relations for North Canadian river upstream and downstream from Canton Dam (after Williams and Wolman, 1984)

Additional data, illustrating the downstream effect of reservoir construction on sediment loads, are presented in Table 2.2. This indicates that maximum differences between natural and regulated sediment loads occur immediately below the dam. Further downstream sediment discharge increases, but it still does not reach the natural, pre-construction, value. On the Missouri river sediment run-off 7km downstream from Gavins Point Dam, which was completed in 1955, is just 1% of the natural sediment run-off during the period 1957-1969, while further downstream, as a result of bed erosion and sediment yield from tributaries, it increases as follows: 314km - 17% of natural load (1957-1973), 585km - 22% (1957-1976), 716km - 24% (1957-1976), 1147km - 30% (1957-1976), 1300km - 34% (1957-1980).

Similarly on the Nile, sediment run-off changed considerably after the closure of Aswan High Dam (table 2.3). Just downstream from the dam, for the period 1968-1982, sediment run-off was 1.6% of the natural load, 173km downstream from the reservoir - 1.9%, 366km - 22%, 551km - 2.5% and at 960km, downstream just before it discharges into the Mediterranean - 3.1%. The increase in sediment run-off between the reservoir and the river mouth changed from $2.2.10^6$t/year to $4.1.10^6$t/year, representing an average of $1.9.10^6$t/year for the period 1968-1982. This change only resulted from bed erosion, since there was no additional inflow. The progressive decrease in annual sediment load in the river reach between the dam and the Delta barrage (Table 2.3) probably results from bed armouring.

An increase in sediment run-off immediately downstream from the dam has been observed at some sites during dam construction and in the first few years of reservoir operation. For example, measurements of suspended sediment loads 120km downstream from Kaunassky hydro-electric power station on the Neman river show different relations between the silt charge and water discharge pre and post dam construction (fig. 2.11). During the construction period a considerable increase in silt charge is observed, nearly double that under natural conditions. This increase is probably connected with the construction works at Kaunassky dam and the use of hydraulic excavators when laying the dam foundations on the bed of the river.

In a number of cases decreased water levels below the dam, due to flood attenuation and bed degradation, can lower the base level of tributary streams. This can rejuvenate them and increase their sediment yield to the main river. For example, on the Don downstream from Tzimlianska hydro-electric power station the base level of a tributary, the North Donetz, was lowered increasing the slope of the lower reaches of the river by a factor of between 5 and 10. This precipitated bed erosion and a high sediment input to the Don. Much of this material is deposited at the tributary junction due to the reduced transport capacity of the main river.

TABLE 2.2 Downstream effects of dams on sediment run-off

River	Reservoir	Year of dam closure	Distance from the dam (km)	Sediment Load: before dam closure (10^6t/yr)	Sediment Load: after dam closure (10^6t/yr)	after dam closure %	after dam closure period year	Reference
Colorado	Hoover	1936	180	200	10	5	1936-1939	Williams & Wolman, 1984
Colorado	Glen Canyon	1962	186	126	17	13	1963-1972	"
Missouri	Garrison	1953	121	49	9.8	20	1955	"
North Canadian	Canton	1948	5	20.5	0.1	0.5	1949-1960	"
Missouri	Gavins Point	1955	7	121	1.5	1	1957-1969	"
"	" "		314			17	1957-1973	"
"	" "		584			22	1957-1976	"
"	" "		716			24	1957-1976	"
"	" "		1147			30	1957-1976	"
"	" "		1300	320	109	34	1957-1980	"
Danube	Iron Gate I	1970	5	38	5	14	1974	Richter, 1964
"	" "		110	39	13	33	1975-1979	"
"	" "	1964	389	51	31	61	1970-1979	"
Nile	High Aswan	1964	6.5	134	2.2	1.6	1968-1982	Shalash, 1984
"	" "		173	134	2.5	1.9	1968-1982	"
"	" "		366	134	2.9	2.2	1968-1982	"
"	" "		551	134	3.3	2.5	1968-1982	"
"	" "		950	134	4.1	3.1	1968-1982	"

TABLE 2.3 Sediment run-off, pre- and post-construction of High Aswan Dam, on the river Nile at Low Aswan and the Delta Barrage 6 and 96km respectively downstream from High Aswan Reservoir in 10^6 t/year (after Shalash, 1982)

Year	Low Aswan	Delta Barrage	Difference
1923-1963	134	124	-10.0
1964	26.3	51.8	25.5
1965	5.7	16.5	10.8
1966	3.8	9.0	5.2
1967	3.1	14.4	11.3
1968	2.3	3.2	0.9
1969	1.9	3.2	1.3
1970	2.7	4.4	1.7
1971	2.5	4.5	2.0
1972	2.7	4.4	1.7
1973	2.8	5.1	2.3
1974	2.8	5.1	2.3
1975	1.8	5.9	4.1
1976	1.6	3.9	2.3
1977	1.6	4.0	2.4
1978	2.2	4.7	2.5
1979	2.1	4.1	2.0
1980	1.8	3.3	1.5
1981	1.6	3.1	1.5
1982	1.9	2.6	0.7

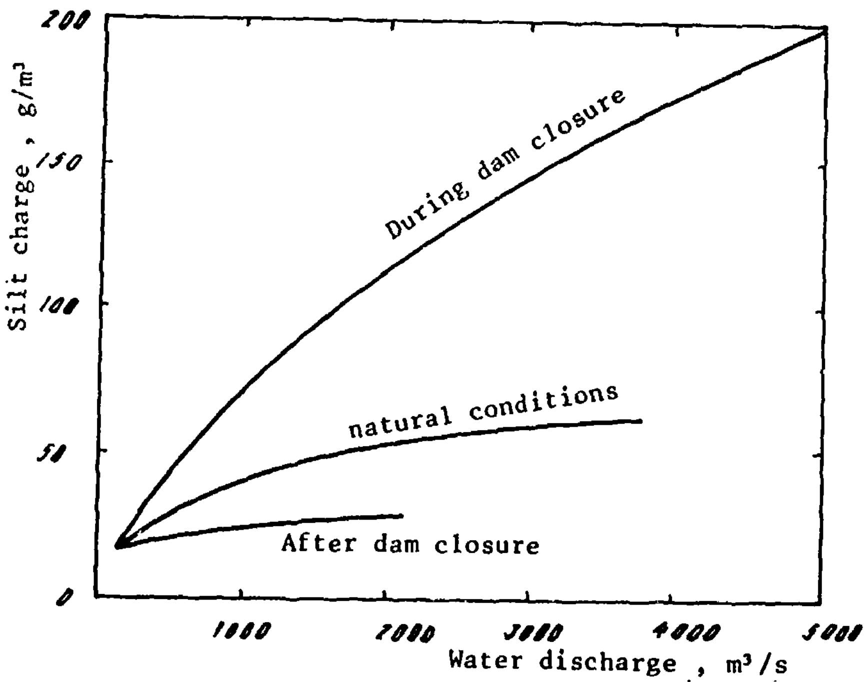


Fig. 2.11 Relation between silt charge and water discharge for the Neman river at a site 120km downstream from Kaunassky dam for natural conditions and for periods during and after dam closure (modified from Veksler and Donenberg, 1983)

A similar problem has occurred on the river Neman downstream from Kaunassky hydro-electric power station where reduced water levels have rejuvenated the rivers Neris, Nevjest, Dubisa and Mituva.

The restoration of sediment discharge downstream from a reservoir to natural, pre dam values is not possible (except in the case when the original reservoir capacity is entirely filled with sediment) for the following reasons:

1. Transport capacity is decreased, as a result of the reduced frequency of high flows, and continues to decreases due to bed erosion;

2. Transport capacity varies with size fraction. When grain sizes are large, sediment discharge for a given flow condition decreases. This is illustrated in figure 2.12 where relations between particle diameter and the transport capacity for suspended sediment (by Karaushev's formulae) and bed load (by Shamov's formulae) are presented. When the diameter of the bed material is 0.04m (as for example, under natural conditions before High Aswan Dam closure), transport capacity is 5kg/m, while in the case of d=0.4mm (after High Aswan Dam closure), transport capacity is 0.5kg/m, i.e. 10 times smaller.

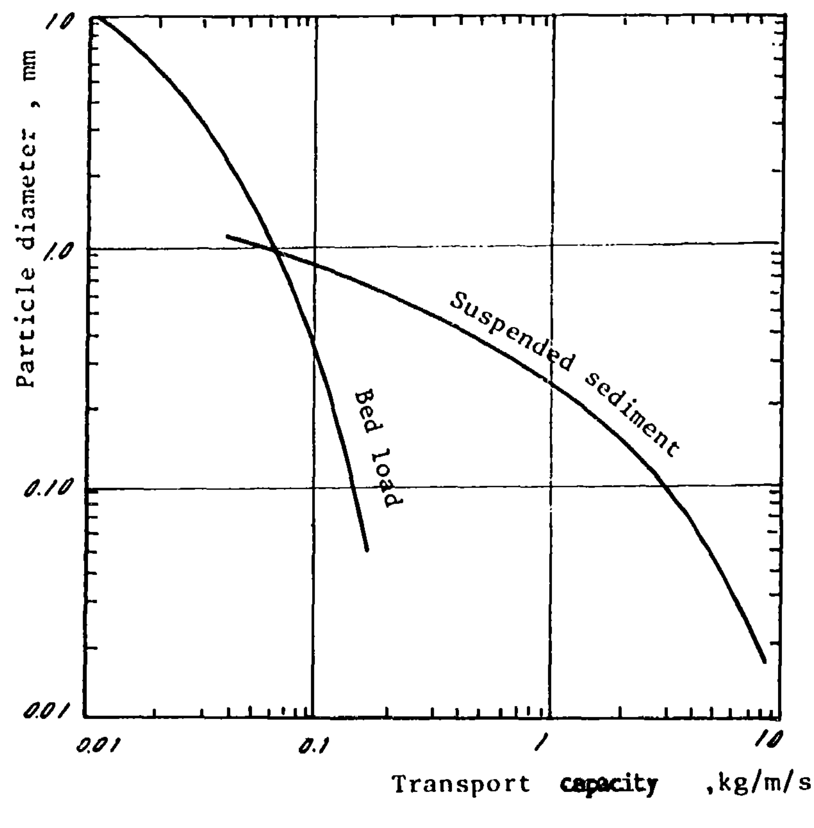


Fig. 2.12 Relation between particle diameter and transport capacity of bed load (by Shamov) and of suspended load (by Karaushev) for a flow depth of 1m, velocity 1m/s and Chezy's coefficient C=40

Figure 2.13 indicates how the size of the suspended sediment increased on the river Danube 109km downstream from the Iron Gate I dam 6 years after dam closure. Under the natural regime the average particle diameter of the suspended sediment at the water surface was 0.09mm, which declined to 0.03mm after regulation. In contrast, the suspended particle diameter near the bed increased from 0.24 to 0.34mm. Subsequent observations indicated further coarsening of the suspended sediment grain size near the bed and a decrease in the size of suspended sediment at the water surface, which increased the depth-suspended sediment concentration gradient (Fig. 2.14).

Therefore, the total suspended load transported by the river downstream from a dam is less than under natural conditions. When erosion is forecast downstream from a dam, full account should be taken of bed armouring when assessing sediment loads after dam closure.

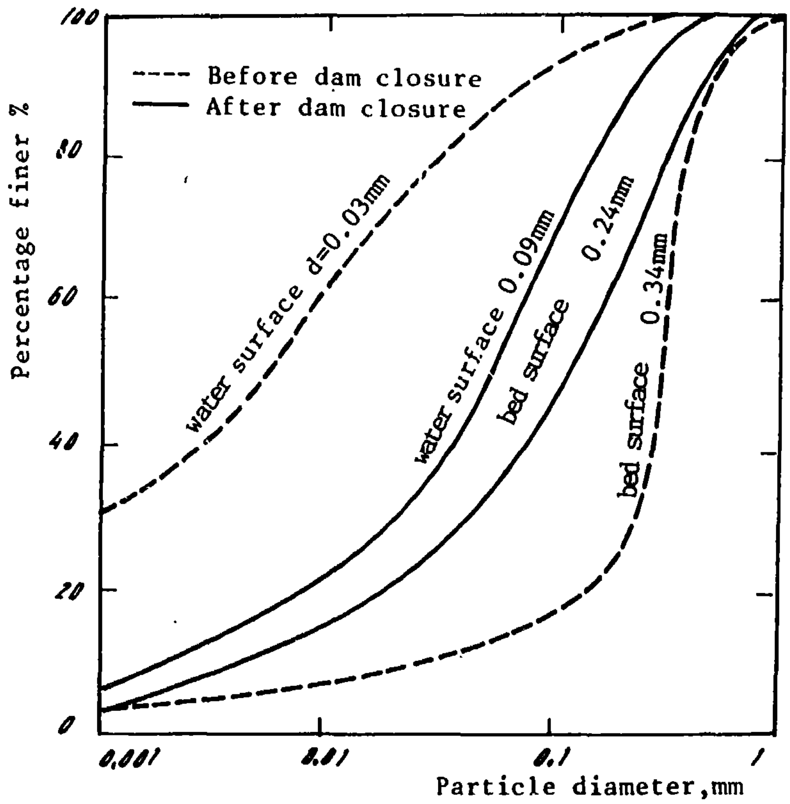


Fig. 2.13 Suspended sediment grain size distributions. Danube river 109km downstream from Iron Gate I dam before and after dam closure (after Pechinov, 1982)

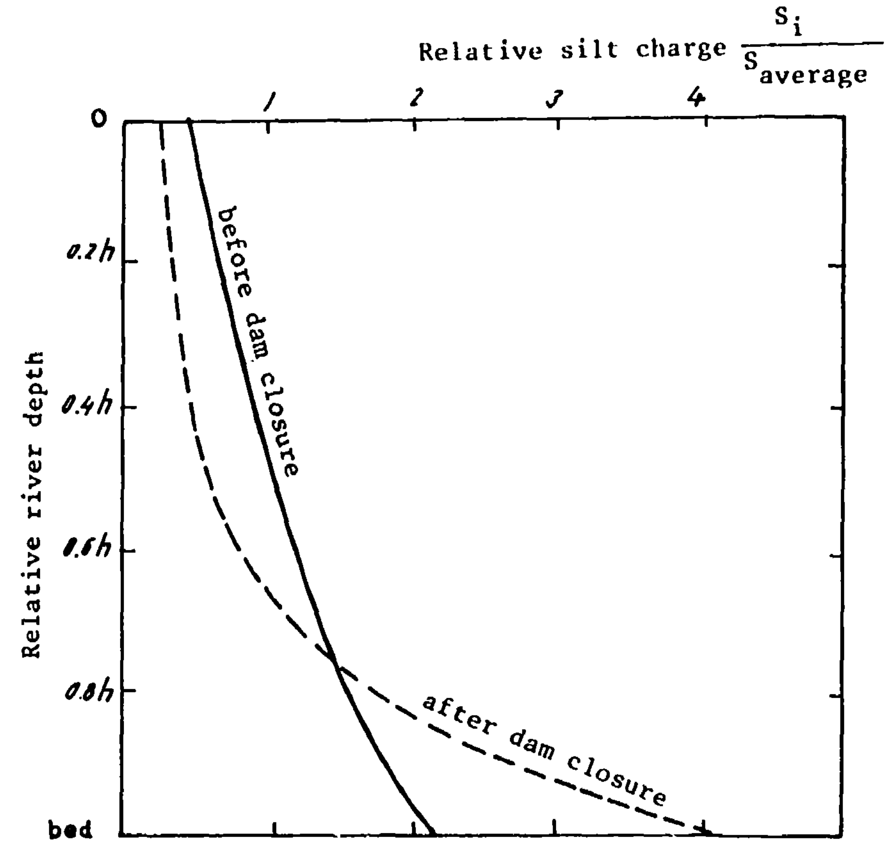


Fig. 2.14 Variation of relative silt charge with depth, river Danube 109km downstream from Iron Gate I dam before and after dam closure (after Pechinov, 1982)

2.2.3 River bed erosion downstream from dams

Erosion downstream from dams may be divided into two types:

1. Local scour - immediately downstream from the dam;
2. General degradation - producing extensive and often progressive lowering of the river bed often over a considerable distance.

The causes of both types of erosion can best be illustrated by considering some typical examples from various countries in the world.

One of the greatest problems with scouring relates to the structural stability of the dam. Degradation can result in damage to and sometimes the failure of bridges, supporting walls, water intakes and pipelines, in adverse operating conditions for hydro-electric power stations and for navigation and in a reduction in ground water levels in flood plains.

Local scour can be prevented, although it is very difficult to take effective measures against degradation.

2.2.3.1 <u>Scour downstream from dams</u> During the construction period and at the start of reservoir operations, major scouring can occur immediately downstream from the dam.

This phenomena is explained by:

1. passage of floods during construction period when river bed is disturbed;
2. incomplete or inappropriate armouring downstream from the dam;
3. reduction in channel width, which leads to a considerable increase in specific water discharge and flow velocity compared to natural conditions.

In order to decrease scour downstream from dams, some form of energy dissipator should be constructed.

Table 2.4 presents data for scour downstream from some large dams in the USSR. This indicates that scour depths in excess of 30m can occur as, for example, downstream from the Kuibishevsky Dam on the Volga.

TABLE 2.4 Scour depths downstream from dams

River	Dam	Beginning of dam operation	Q_{mean}	$Q_{0,1\%}$	Q release		Head	h
					dam	spill-way		depth of scour hole
		(Year)	m^3/s	m^3/s	m^3/s	m^3/s	m	m
Volga	Ivanovski	1937	306	7350	300	7350	11	3.5 - 4.0
	Uglichski	1940	430	11600	1200	11600	11	2.0 - 2.5
	Rybinski	1940	1120	12600	3600	5600	15.5	5.0 - 10.0
	Gorkovski	1950	1690	19800	3280	11820	14.2	7.0 - 11.5
	Kuibishevski	1955	7620	70800	12000	38000	20	3 - 31
	Saratovski	1970	7820	69100	7800	45200	10.6	10
	Volgogradski	1959	7960	62900	16530	30850	20	10 - 20
Don	Tzimilianski	1954	635	17200	1130	22400	22	5 - 6
Neman	Kaunaski	1960	293	3650	760	2340	14.8	0.5
Ob	Novosiirski	1959	1640	15900	2870	11500	15.8	6.5
Daugava	Rijskii	1975	640	10200	3650	7800	15.4	6.0
Saalach	Reichenhall	1913	39					15.0
Isar	Dingolging	1957	167					19.0
Iller	Unterbalzhein	1969	55					4.5
Wertah	Schwabmünchen	1956	20					4.5

In the first few years following the construction of Kuibishevsky dam in 1955, serious scouring occurred downstream from the hydro-electric power station (Veksler and Donenberg, 1983). The scour hole was partially filled with large stones, total volume 200,000m^3, which prevented further erosion. Scoured material was initially deposited just downstream from the scour hole before subsequent dispersal downstream (fig. 2.15). Deposited scour material initially caused water levels to be increased by up to 55cm (fig. 2.16). Subsequently erosion occurred and after 12 years the natural bed elevation became re-established.

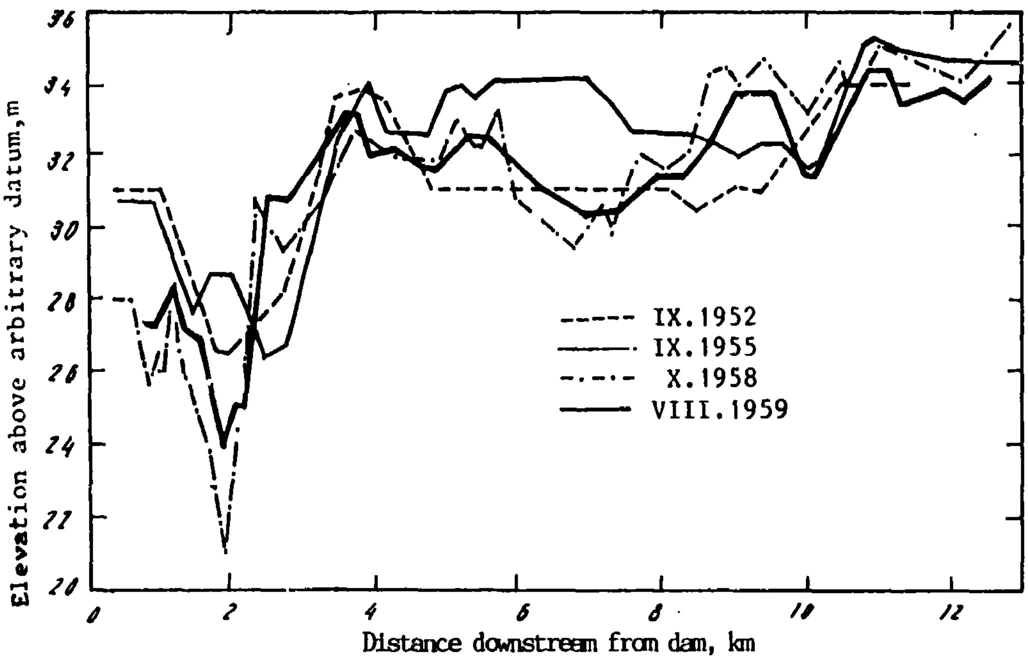


Fig. 2.15 Change in longitudinal profile of the river Volga downstream from Kuibishevski dam (1952-1959) (modified from Veksler and Donenberg, 1983)

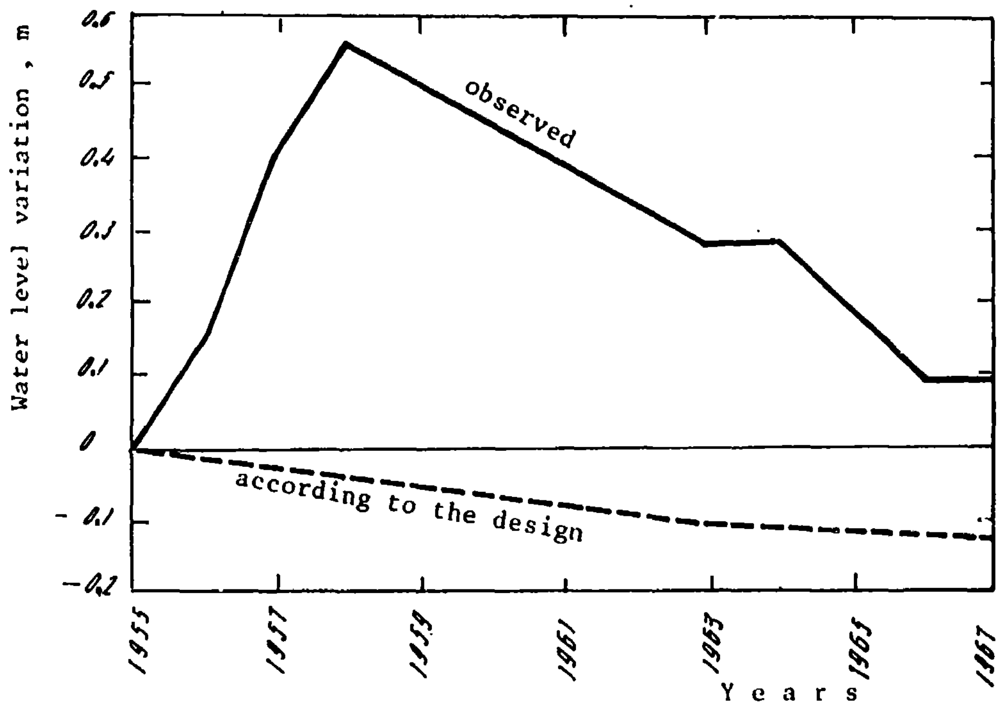


Fig. 2.16 Change in water levels on the river Volga downstream from Kuibishevski dam in the first few years after dam closure due to the deposition of scoured material (after Veksler and Donenberg, 1983)

Considerable scouring has also been observed downstream from Volgogradski dam on the Volga in the first few years following dam construction. Scour depths of up to 18m were observed downstream from the power station and 20m downstream from the spillway with computed depths comparing well with observed values. These predictions were obtained using the Rossinsky and Kuzmin method, as outlined in chapter 8. About 12×10^6 m^3 of bed material was eroded, out of a total of 26.5×10^6 m^3 injected near the dam, over a period of 5 - 6 years.

A more complex scour pattern occurred on the Volga downstream from Gorkovskii dam (completed in 1956). Deposition of scoured material during and immediately after dam construction is shown on figure 2.17. This caused a downstream increase in water levels in the period 1956-1958 (Fig. 2.18). Subsequently, as a result of degradation downstream from the dam and flood releases from the reservoir, the deposited material was quickly eroded (Fig. 2.17), leading to a decrease in water levels to values corresponding to the long term mean values. By 1982 the water level was still lower than the natural level in the pre-dam period (Fig. 2.18).

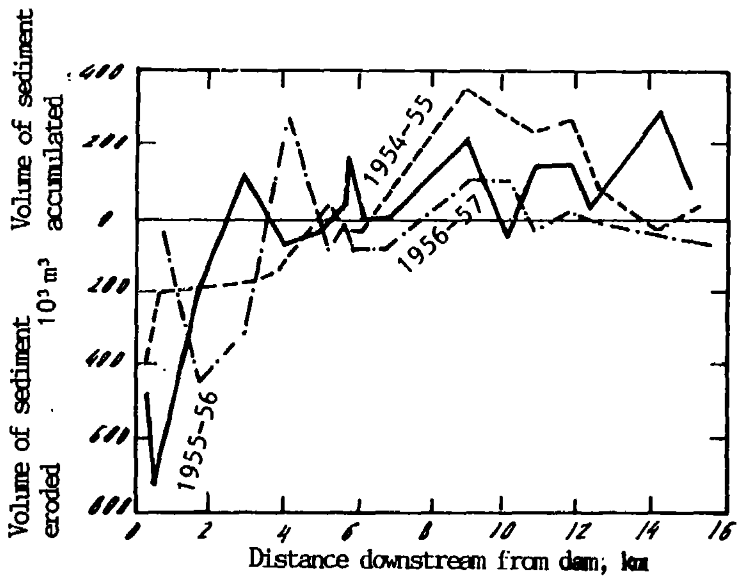


Fig.2.17. Volume of sediment eroded and deposited downstream from Gorkovski dam, river Volga, for the period 1954-1957 (after Veksler and Donenberg, 1983)

2.2.3.2 <u>General degradation of the river bed</u> In order to evaluate the influence of engineering structures on river bed erosion, consideration should be given to the natural instability of the channel. Data in table 2.5, based on information from Williams and Wolman (1984) and expanded with data from other authors, indicates that long-term rates of change in bed elevation are very low - in the range from -0.0005 to + 0.03m/year. Although the rates of change are small, over long time periods they can produce relatively large changes in bed elevations. For example, aggradation on the Saskatchevan river raised bed levels by 7.2m over 2400 years and by 0.93m over 31 years on the Colorado; while degradation on the Klarälven river reduced the bed elevation by 49m over a period of

7000 years, by 3.26m over 815 years in the Red Creek river, by 2.75m over 250 years in the Beatton river .

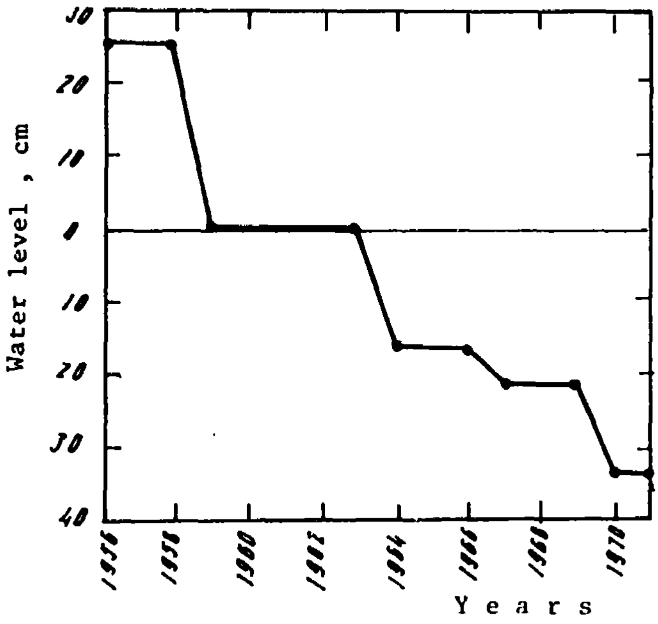


Fig.2.18 Change in water level downstream from Gorkovski dam, river Volga, for the period 1956-1980 (after Veksler and Donenberg, 1983)

Short term changes in bed elevation due to the passage of individual floods can be significant; up to 24m increase in bed elevation over a 3-day period on the Waino river (New Zealand), and 9m on the Yellow river (China) within 12 hours (table 2.5).

The data show that major changes in river bed elevations are uncommon, while small long term changes are relatively insignificant with regard to river engineering and management activities.

Studies by Glazik (1964), Bauer (1963), 1965), Schmuterer (1952), Bensing (1966) Pichi (1958) and Eschweiler (1952) illustrate the effect of engineering structures and training works on bed levels on the Elbe, Danube and Rhein (table 2.6). These indicate the change of water levels corresponding to a particular discharge value.

On the Elbe water levels corresponding to the standard discharge decreased at all the hydrometric stations from river km 2.1 to river km 353.8 (Glazik 1964). The lowest mean annual decrease (0.3cm/year) is at Schöna for the period 1886 - 1961. The largest mean annual decrease, 2.1cm/year for the period 1892 - 1961 and total change 142cm for the same period, was observed at Torgau (Table 2.6).

Changes in water levels on the Danube corresponding to the long-term mean annual discharge for 13 sites between river km 2459 and river km 1949 were investigated by Bauer (1963, 1965) and Schmutterer (1952). Reduced water levels were observed at twelve stations (1900-1960) with a maximum rate of decrease of 3.0cm/year at Ingolstadt. Only at Kelheim was an increase in water level, 23cm, observed (Table 2.6).

TABLE 2.5 Changes of river beds unaffected by manmade works *

No.	River	Country	Long-term changes: period year	Long-term changes: change m/year	Short-term changes: period	Short-term changes: change m	Reference
1.	Colorado	USA	31	+0.03			Cory, 1913
2.	Yellow	China	about 30	+0.03			Todd and Eliassen, 1940
3.	Alexandra-North Saskatchewan	Canada	2400	+0.03			Smith, 1972
4.	Kodori	USSR	32	+0.03			Mandych and Chalov, 1970
5.	Last Day Gully	USA	11	+0.006			Emmett, 1974
6.	Arroyo de Los Frijoles	USA	6	+0.01			Leopold and others, 1966
7.	Brahmaputra	Bangladesh			6 months	+8	Coleman, 1969
8.	James	USA			several hours	+1.5	Williams and Guy, 1973
9.	Van Duzen	USA			about 3 days	+3	Kelsey, 1977
10.	Little Larrabee Creek	USA			about 3 days	+2.4	Kelsey, 1977
11.	Trinity	USA			"	+3.4	Ritter, 1968
12.	Waino	New Zealand			"	+24	Gage, 1970
13.	Centre Creek	" "			8 months	+0.5	O'Loughlin, 1969
14.	Castaic Creek	USA	100	-0.01			Lustig, 1965
15.	Red Creek	USA	about 815	-0.004			La Marche, 1966
16.	Beatton	Canada	250	-0.011			Hickin and Nanson, 1975
17.	Lena	USSR	20	-0.0005			Borsuk and Chalov, 1973
18.	Klarälven	Sweden	about 7000	-0.007			de Geer, 1910
19.	Trinity	USA				-0.5	Ritter, 1968
20.	Wills Cove	USA			several hours	-3	Williams and Guy, 1973
21.	Yellow	China			about 12 hours	-9	Todd and Eliassen, 1940
22.	Centre Creek	New Zealand			8 months	-0.5	O'Loughlin, 1969
23.	Klarälven	Sweden			about 1 month	-4.7	de Geer, 1910

* "+" - aggradation, "-" - degradation-

TABLE 2.6 Water level changes on the Elbe, Danube and Rhein rivers.

River	Point	River km	Period	Decrease (cm) totally	Decrease (cm) per year
Elbe	Schöna	2.1	1886-1961	21	0.3
	Dresden	55.6	1886-1961	77	1.0
(Glazik,	Strehla	115.1	1886-1960	60	0.8
1964)	Torgau	154.6	1892-1961	142	2.1
	Aken	274.7	1904-1961	41	0.7
	Barby	293.4	1883-1961	75	1.0
	Magdeburg	332.7	1923-1959	64	1.8
	Wittenborge	454.6	1904-1960	17	0.3
	Lenzen	484.6	1884-1953	43	0.6
	Darchau	535.8	1887-1952	102	1.6
Danube	Ingolstadt	2459	1900-1960	177	3.0
	Vohburg	2443	1900-1960	57	1.0
(Bauer,	Newstadt	2432	1900-1960	19	0.3
1964,	Kelheim	2424	1900-1960	23 increase	
1965)	Niederwinzer	238	1900-1960	58	1.0
	Regensburg	2379	1900-1960	73	1.2
Schmuterer	Schwabelweis	2376	1900-1960	62	1.0
(1952)	Straubing	2321	1900-1960	41	0.7
	Obernzell	2208	1936-1949	20	1.4
	Aschach	2262	1936-1949	32	2.3
	Hollenburg	1994	1936-1949	15	1.0
	Zwentendorf		1936-1949	34	2.4
	Greifenstein	1949	1936-1949	20	1.4
Rhein	Karlsruhe	362.3	1856-1950	30 increase	
	Speyer	400.6	1856-1950	100	1.0
(Bensig,	Mannheim	424.9	1856-1950	170	1.8
1966)	Worms	443.4	1856-1950	115	1.2
(Pichi,	Oppenheimer	481	1856-1950	105	1.1
1958)	Meinz	498	1856-1950	90	1.0
(Eschweiler	Bingen	528	1856-1950	35	0.4
1952)	Kaub	546	1856-1950	0	0
	Koblenz	592	1856-1950	60	0.6
	Köln		1820-1950	54	0.4
	Düsseldorf	744	1820-1950	112	0.9
	Ruhrort	781	1820-1950	213	1.6
	Wesel	814	1820-1950	248	1.9
	Rees	837	1820-1950	181	1.4
	Emmerich	852	1820-1950	133	1.0

Similar studies were carried out on the Rhein by Bensing (1966), Pichi (1958) and Eschweiler (1952), covering 15 stations between river km 362 and river km 852. Decreased water levels were observed at 14 of the stations, the largest decrease, 248cm, was at Wesel (1820 - 1950), giving an average decrease of 1.9cm/year.

Downstream from dams on alluvial rivers degradation normally occurs, regardless of the degree of run-off regulation and its associated influence on sediment transport capacity, due to sediment being trapped by the reservoir. As transport capacity below the dam exceeds sediment supply, erosion results.

However, in cases where most of the streamflow is diverted, aggradation and a reduction in channel width occurs, due to sediment inflow from tributaries exceeding the transport capacity of the river. According to Nenov (1985) this phenomenon is observed downstream from approximately 50% of the reservoirs in Bulgaria. A typical example is the Iskar reservoir on the river Iskar (Bulgaria), where all the stored water is diverted for over 30km through canals for power production and for irrigation and water supply purposes. The river below the reservoir is practically non-existant while further downstream its size relates to the additional streamflow from the drainage area. Most of diverted flow (70%) is returned along a 20km length of river 50km downstream from the dam, where the river approximates its natural state.

As a result of river regulation, flow levels are reduced which lowers the base level for the tributaries. This causes severe erosion near the mouth of the tributaries and deposition of eroded material on the bed of the main river, as regulation has reduced its transport capacity. In certain circumstances, as for example on the Rio Grande downstream from Elephant Butte Dam, bed levels have been raised over a distance of 265km due to sediment supplied by the tributaries.

There are also cases, especially on rivers where the bed consists of coarse gravel and boulders, where releases are below the threshold discharge for bed material transport. In such cases no erosion occurs.

Generally, however, considerable degradation occurs on alluvial rivers downstream from dams resulting in the lowering of the river bed over long distances.

2.2.3.3 <u>Examples of bed degradation downstream from dams</u>
Immediately after dam construction, significant degradation usually occurs close to the dam. After a certain period of time erosion rates decline and the process progresses downstream.

In the case of uniform bed material, river bed lowering should be a maximum adjacent to the dam and gradually decrease downstream. This leads to a reduction in the longitudinal slope of the river (Fig. 2.19). Such occurrences are comparatively rare.

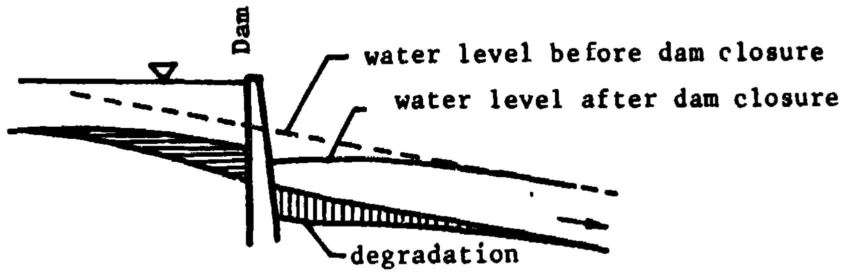


Fig. 2.19 Erosion downstream from dams

More often variations in the calibre of the bed material influence the degree and extent of erosional activity. Other factors controlling degradation include presence of rapids, tributaries, storage or releases that affect the transport of bed material and changes of base level. Consequently every river is likely to react differently to dam construction.

Tippner's studies (1973) showed that erosion downstream from dams does not always follow the classic scheme given in figure 2.19. He studied erosion downstream from the Gerstheim dam on the Rhein (from km 274 to km 310). Immediately after dam construction (1966-67), scouring occurred downstream from the dam; deposition in the reach to km 300, and erosion further downstream (fig. 2.20). Between 1967-1968 degradation progressed downstream to km 285 with aggradation further downstream. In the third period, 1968 to 1969, virtually the whole reach studied was eroding, except for a section of the river located near the dam. The observed pattern of instability downstream from the dam is due to local differences in channel slope and bed material size which influence transport capacities. The general tendency is for the erosion downstream from dams, inspite of some aggradation during the first few years after dam closure.

Further examples of erosion downstream from dams are presented by Williams and Wolman (1984). Erosion on the Smoky Hill river downstream from Kanopolis Dam most closely approaches the theoretical model, at least for the first 15km downstream from the dam (fig. 2.21). Longitudinal profiles of the Colorado river indicate that degradation downstream from Parker Dam has significantly affected a reach 60km long. Maximum erosion occurred in the first four years following dam closure (fig. 2.22).

Changes in channel profile downstream from Fort Randall Dam on the Missouri River, although generally tending to flatten with time, vary considerably between cross sections (fig. 2.23), due to Gavins Dam increasing river levels 3 years after Fort Randall Dam was closed.

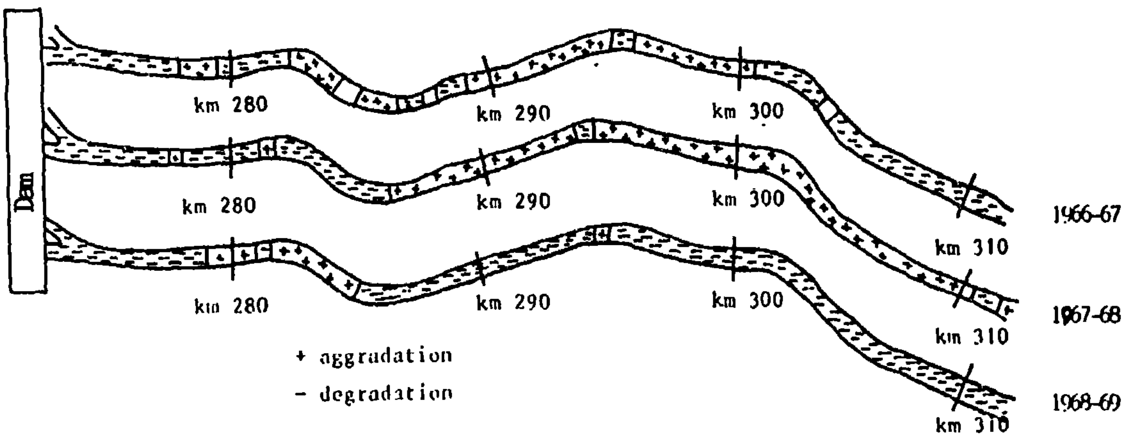


Fig.2.20 Location of eroding and depositing reaches of river Rhine downstream from the Gerstheim dam for the period 1966-1969 (after Tippner, 1973)

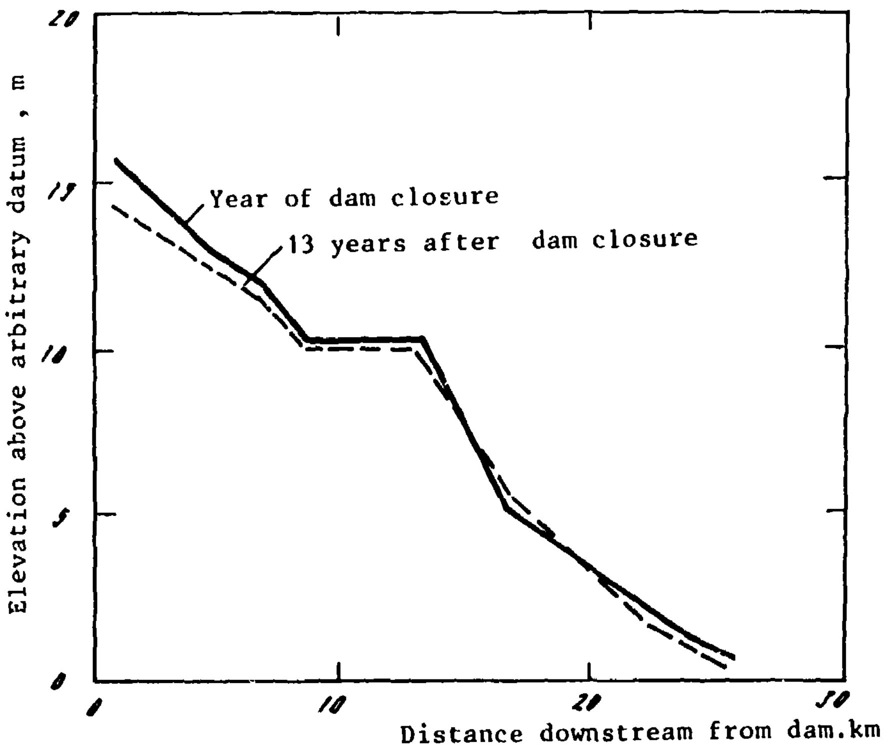


Fig.2.21 Longitudinal profile of the Smoky Hill river downstream from Kanopolis dam at dam closure and 13 years later (Williams and Wolman, 1984)

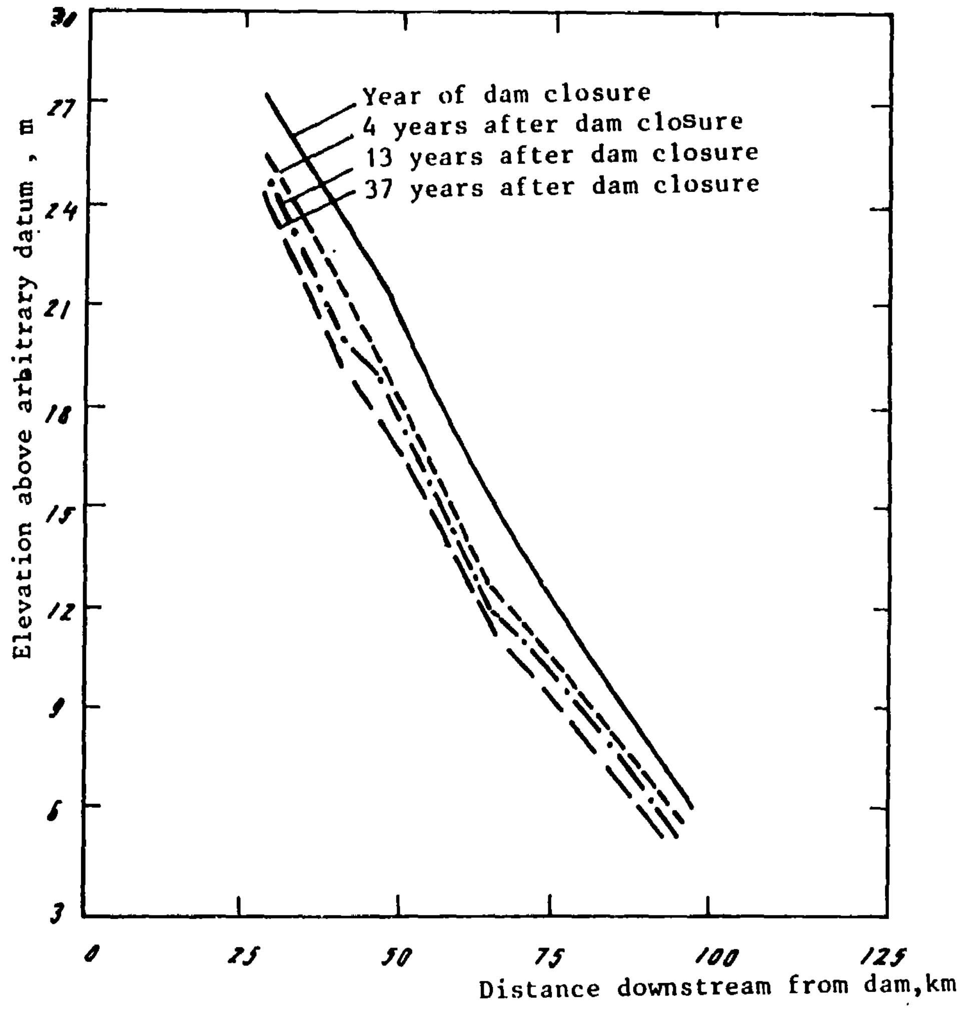


Fig.2.22. Longitudinal profile of the Colorado river downstream from Parker dam at dam closure and 4, 13 and 37 years later (Williams and Wolman, 1984)

Degradation downstream from Glen Canyon dam on the Colorado illustrates the usual progressive reduction in the slope of the river after dam closure (fig. 2.24). Three years after closure maximum scour occurred near the dam, while after nine years this had migrated to a point 15km downstream. Erosion had effectively ceased near the dam due to bed armouring, and this caused the downstream translation of the erosion process. It should be noted, however, that slopes can be increased locally, for example between 16 and 20km from the dam.

The scale and rate of river bed erosion depends on local conditions and particularly on the erosional resistance of the bed material as illustrated by the Sar Daria river downstream Farhadski dam (fig. 2.25). For 21km downstream from the dam the river flows through conglomerates with the result that virtually no degradation occured in the period 5 to 9 years after dam closure. Water diversions from the reservoir and its return to the river 20km downstream from the dam also reduces the transport capacity of the river. Further downstream in the same period considerable degradation was observed which reduced the slope of the river from 0.00018 to 0.00012 over a 250km reach.

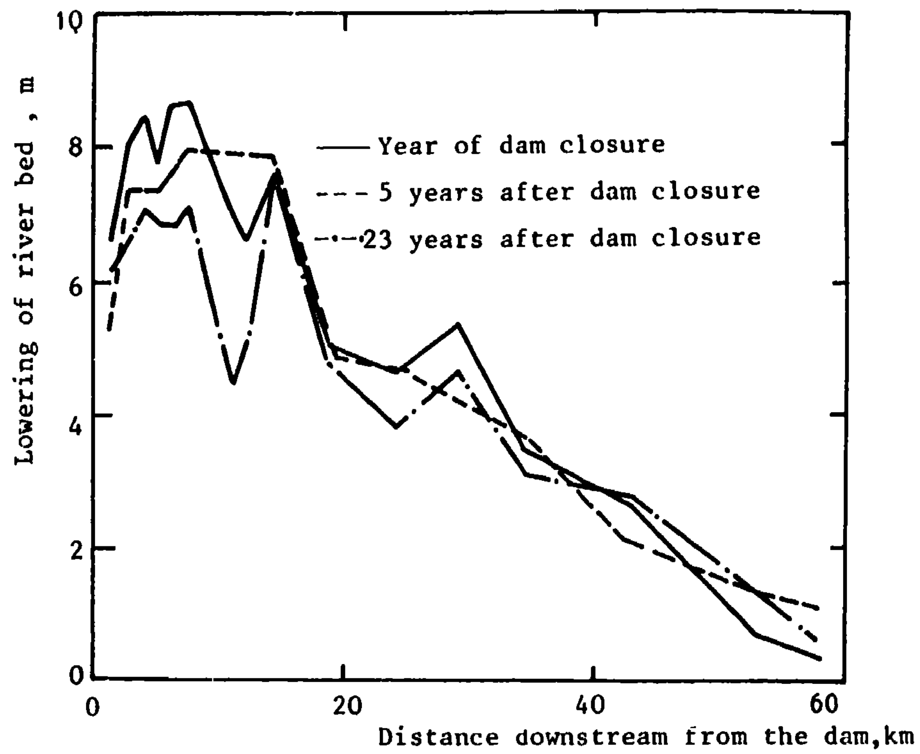


Fig.2.23 Longitudinal profile of the Missouri river downstream from Fort Randall Dam at dam closure and 5 and 23 years later (Williams and Wolman, 1984)

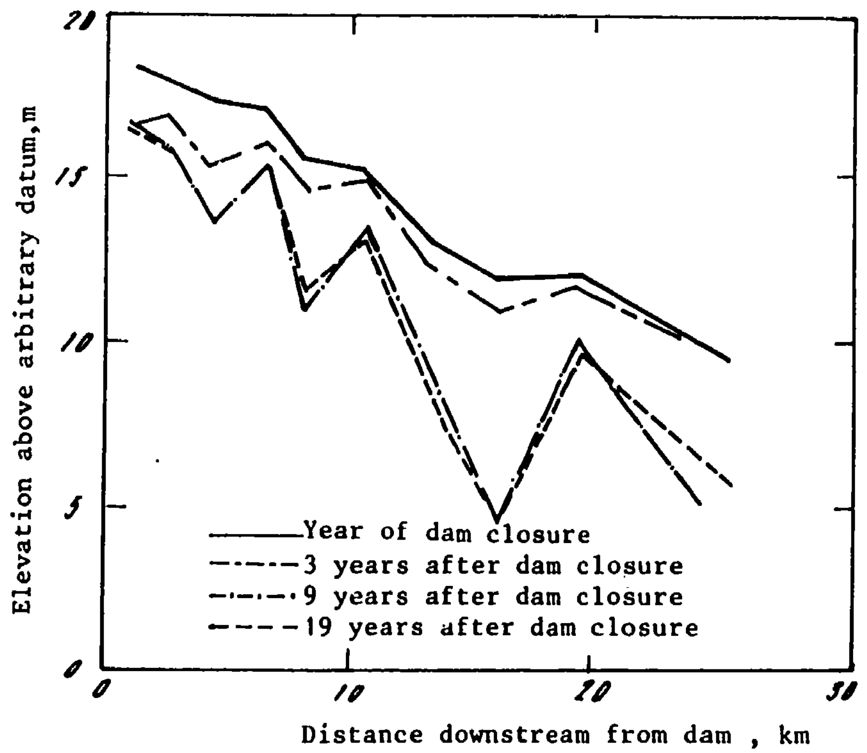


Fig.2.24 Longitudinal profile of the Colorado river downstream from Glen Canyon Dam at dam closure and 3,9 and 19 years later (Williams and Wolman, 1984)

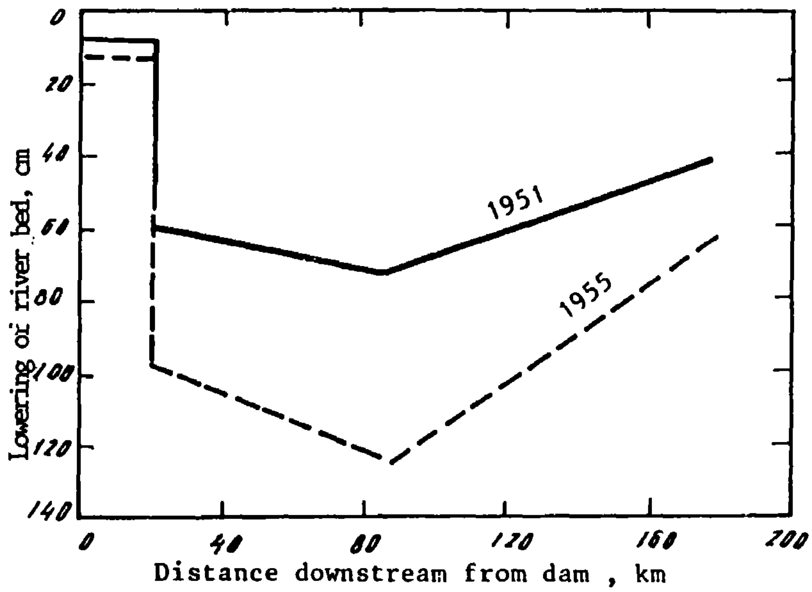


Fig.2.25. Lowering of the river bed downstream from Farhadski dam, river Sar Daria, (1947 - year of dam closure) between 1951 and 1955 (after Altunin, 1958)

Degradation on the Danube downstream from Iron Gate I dam during the first 10 years following dam closure was determined on the basis of a sediment budget (Fig. 2.26). The reduction in bed levels adjacent to the dam was 0.6-0.7m and degradation was observed for 300km below the dam. It should be noted that 943km downstream from the dam, at the Danube delta, islands (formed of fine sediment) are being eroded. This probably results from Iron Gate I dam trapping a large proportion of the suspended sediment run-off from the catchment.

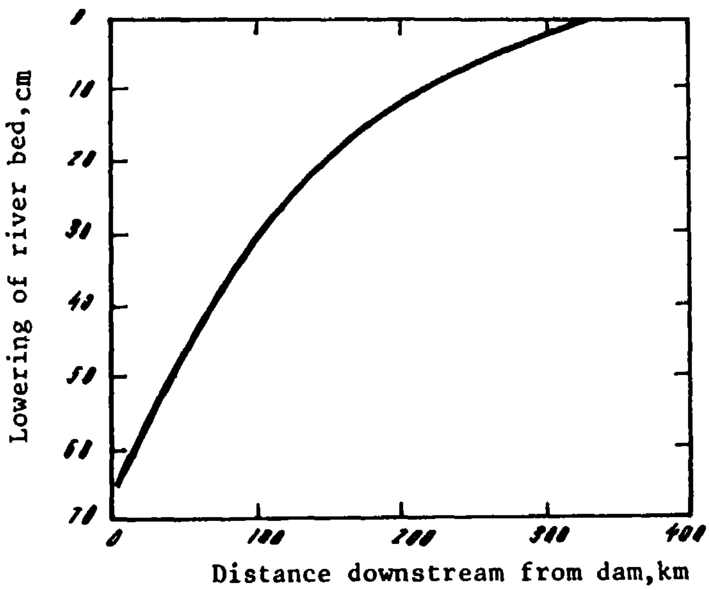


Fig.2.26. Lowering of the river bed downstream from Iron Gate I dam, river Danube, in the 10 years following dam closure

As a result of dam closure on the Visla river in Vlotzlavk (Fig.2.27) both bed and bank erosion has occurred. Increased depth resulted from the erosive power associated with diurnal changes in water levels of up to 3m. As releases are made through a turbine, a rapid increase in water surface slope occurs which, within minutes, is 2-4 times greater than the long term mean annual value prior to dam construction. This indicates that an increase in stream power can cause degradation and lowering of the river bed. River bed erosion in the initial period following dam closure is most active just downstream from the dam and subsequently progresses downstream as an "erosive wave". Babinski 1982) determined that in 4 years following reservoir construction more than $15x10^6$ m^3 of bed material was eroded from a 10km reach downstream from the dam which lowered the river bed by an average of 0.43m. 67m below the dam the river bed was lowered about 2.5m (fig.2.27). The sand and gravel component of the eroded material ($11x10^6$ m^3) was deposited in the reach 10 to 18km below the dam. Throughout the 4 years period the bed elevation in this reach increased by an average of 0.21cm. As a result of degradation the bed material became coarser; the mean diameter increasing by 10 times 200m downstream and 1.5 times at 4km.

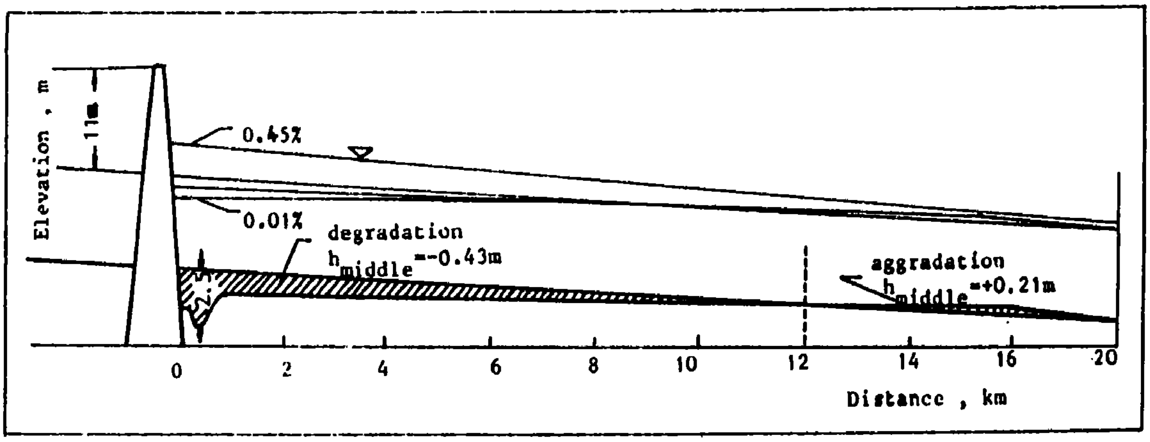


Fig.2.27 Degradation downstream from Vlotzlavski dam, river Visla, over a 4-year period following dam closure (after Babinski, 1982)

The progressive degradation of the Wertach river at Schwabmünchen downstream from a series of dams and weirs has been studied by Bauer and Burz (1968) over a 25 year period. Results showed that the bed of the river has been lowered about 8m (fig.2.28). This illustrates the role of the erosional resistance of the bed material as well as the frequency of flows in excess of $30m^3$/sec.. Thus when the river bed is lowered to the resistant clay layer, the mean annual degradation of the river bed is 19cm/year (table 2.7) (inspite of the large number of days with discharges more than $30m^3/s$). In constrast when fine sand is exposed on the river bed, irrespective of the reduction in the number of days with discharges more than $30m^3/s$, the erosion rate increased to 96cm/year.

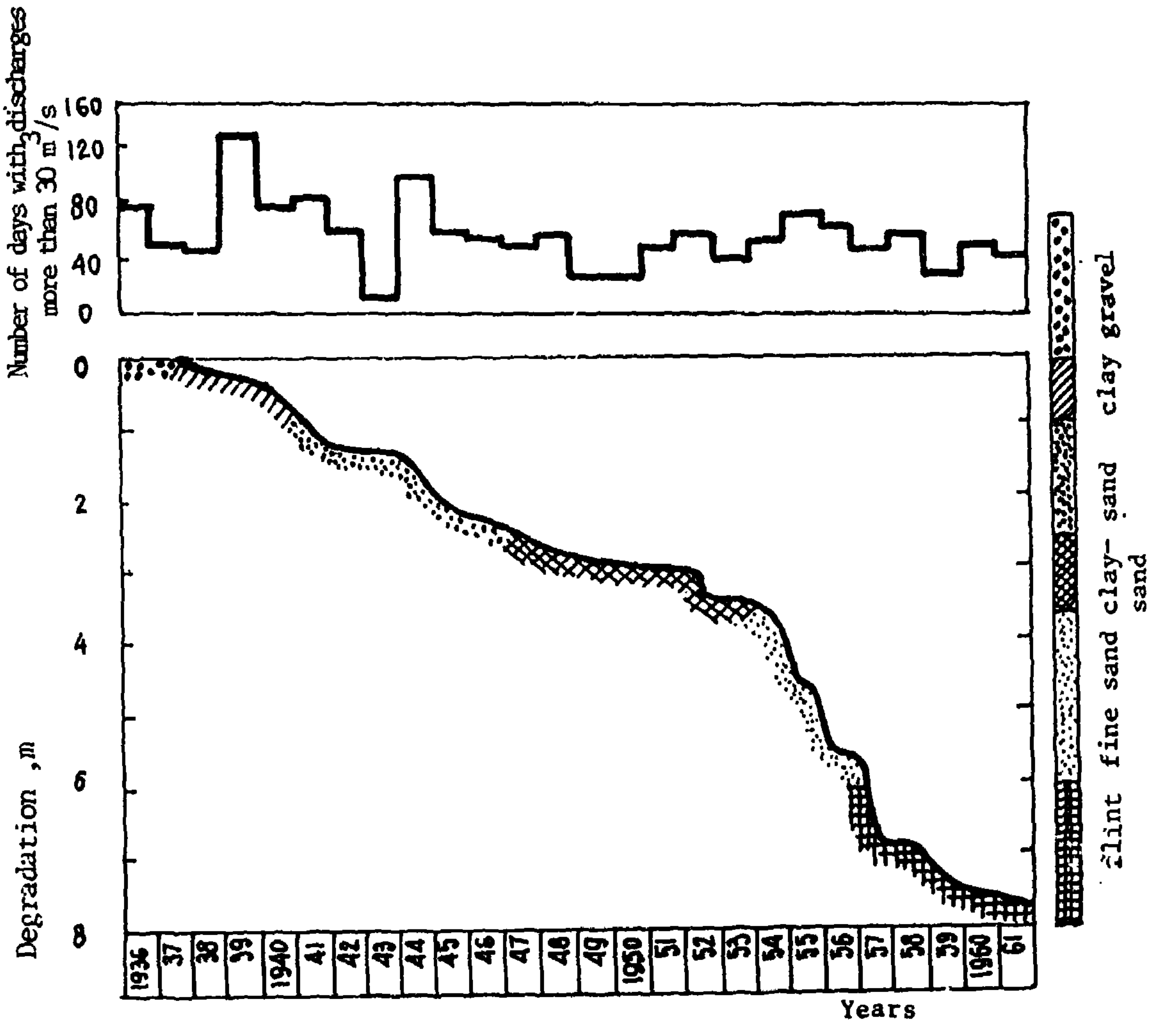


Fig.2.28. Degradation of river bed at Schwabmünchen, river Wertach, (modified from Bauer and Burz, 1968)

TABLE 2.7 Degradation on the Wertach river at Schwabmünchen

	Thickness of the layer (cms)	Duration of degradation (years)	Rate of Lowering of bed (cm/year)	Average number of days/year with water discharges more than 30m^3/sec
Gravel	200	30.0	7	56
Clay	65	3.5	19	86
Sand	160	6.0	27	62
Clay-sand	100	7.5	13	43
Fine sand	240	2.5	96	64
Flint	160	4.5	36	42

The influence of streamflow variability, caused by the operation of hydro-electric power stations, on degradation processes, is illustrated by changes that have occurred on the river Arda downstream from Studen kladenetz dam. The regulating potential and trap efficiency of the reservoir are considerable. The power station operates almost exclusively to supply peak demand. A typical daily discharge hydrograph at a point 10km downstream from the power station is shown in figure 2.29. In this example discharge changes from 1 to 130m^3/s over a very short period of time and produces a rapid increase in the velocity of flow. The high velocities during maximum releases have considerable erosive potential. As a result of degradation, the water level corresponding to the mean annual flow continuously decreased in the period 1958-1970 (fig.2.30). Since 1970 degradation has not been observed, probably due to armouring of the river bed by a protective layer of coarser material. It is possible, when large discharges overtop the dam, that the protective layer will disintegrate allowing further erosion until the armour layer reforms.

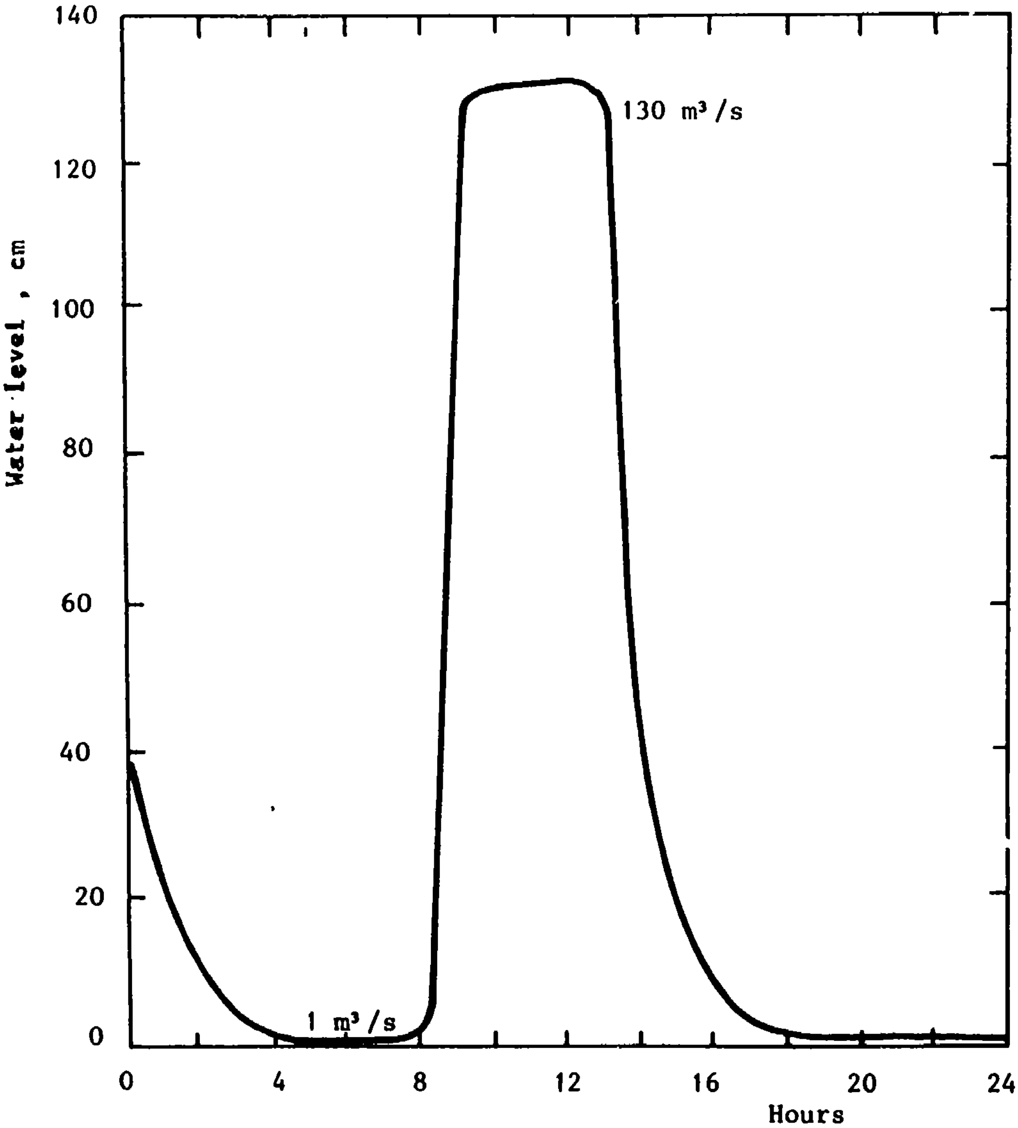


Fig.2.29. Regulated water levels and water discharges on the river Arda 10km downstream from Studen kladenetz dam on 30 August 1984.

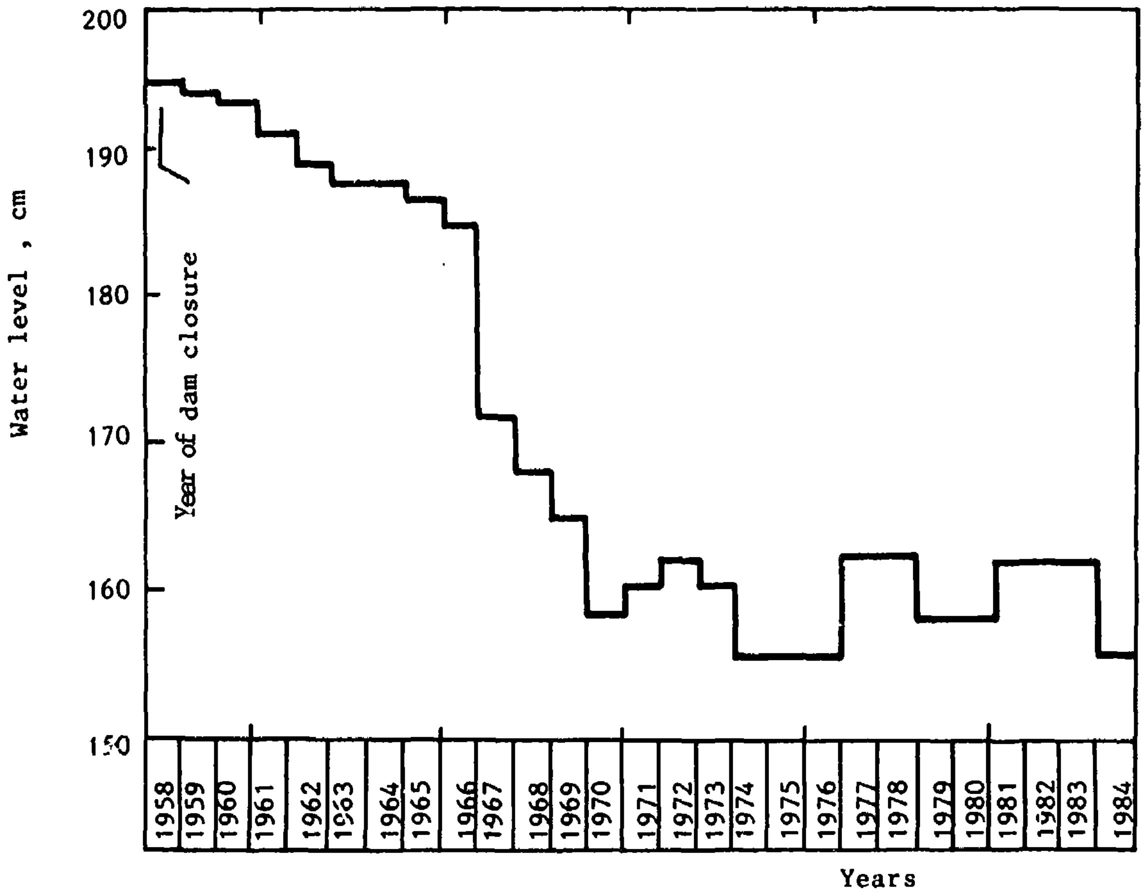


Fig.2.30. Changes in water level corresponding to the annual mean flow for the Arda river 10km downstream from Studen kladenetz dam for the period 1958-1984.

Another example of degradation resulting from a reduction in sediment supply is presented by Gupta et.al. (1967). Bed load transported by the Ratmau river during flood periods is deposited in the Ganga Canal. The relatively sediment free water discharged below the spillway has eroded the river bed over a 120 year period (1845-1966) by as much as 10m over a distance of 15km (fig.2.31).

2.2.3.4 Maximum degradation of river bed Systematic observations of river bed lowering downstream from dams or engineering structures are not always made, or results that are obtained are not always published. The data given in table 2.8 for maximum degradation downstream from reservoirs are not complete but they do give an indication of the range of possible values. It indicates that degradation downstream from some dams is considerable: 7.5m in the first 13 years downstream from Hoover Dam on the river Colorado, 7.3m in 9 years on the Colorado below Glen Canyon Dam, 6.1m in 30 years downstream from Davis Dam on the Colorado, 5.6m in 7 years downstream from Tedjenski dam on the river Murgab and 4.6m in 47 years downstream from Reichenhall Dam on the Saalach river.

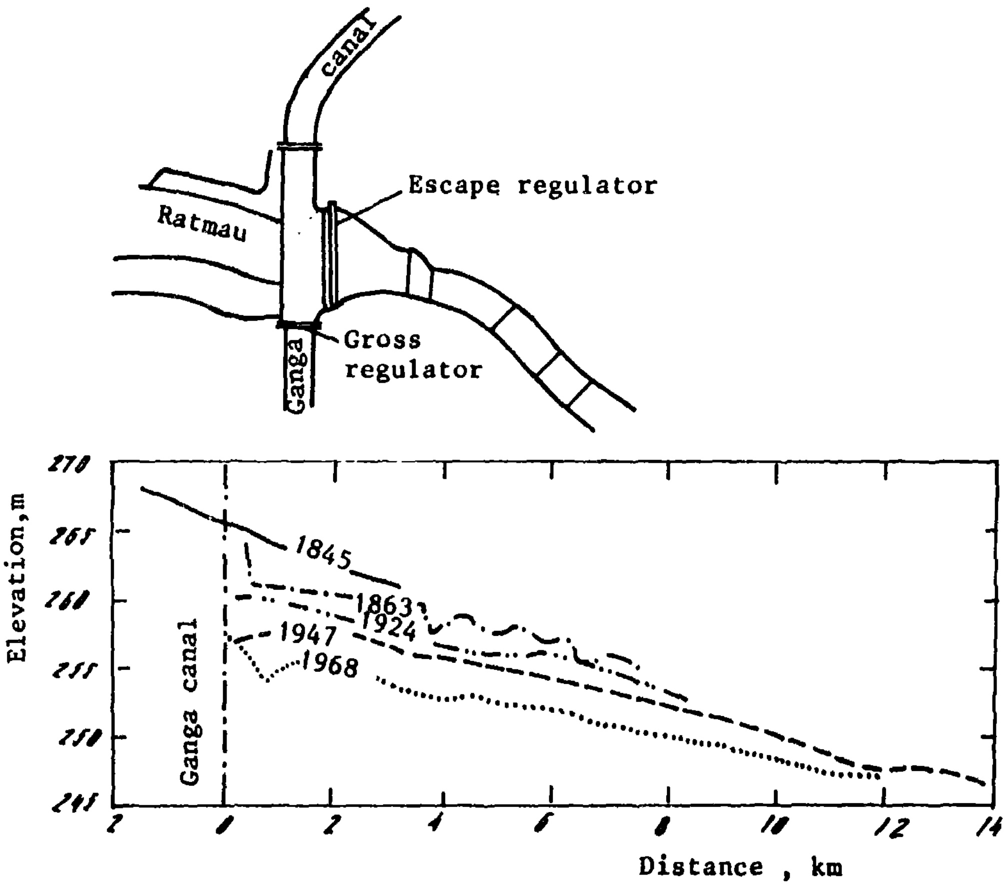


Fig.2.31. Changes in bed elevation on the river Ratmau 1 and 3 km downstream from the spillway on the Ganga Canal for the period 1845-1968 (after Coldwell, 1947)

TABLE 2.8 Maximum Degradation of river bed downstream from dams

River	Dam	Depth of maximum Degradation (m)	Period (year)	Bed material	Source of data
S. Canadian	Conchos	3.1	10	sand, gravel	Hathaway, 1948
Middle Loup	Milburn	2.4	16	sand	USBR, 1945*
Missouri	Fort Peck	1.8	36	sand, gravel	Hathaway, 1948
Colorado	Hoover	7.5	13	sand, gravel	USBR, 1967
Colorado	Davis	6.1	30	sand, gravel	"
Colorado	Parker	4.6	27	sand, gravel	"
Colorado	Imperial	3.1	18	sand	"
Salt Fork	Great Salt Plains	1.0	9	sand	Coldwell, 1942
Red	Denison	3.0	16	sand	Williams & Wolman, 1984
Manistee	Junction	3.7	12	sand, clay	Lane, 1955
An Sabee	Foote	1.5	15	clay	"
Saskatchewan	Squaw Rapids	1.2	13	sand	NHC, 1976 **
Cheyenee	Angostura	1.5	16	sand	USBR, 1967
S. Saskatchewan	Dietenbaker	2.4	12	sand	Galay et al., 1982
Yellow	Sanmexia	4.0	4	fine sand	Li et al., 1980
Colorado	Glen Canyon	7.3	9	fine sand	Williams & Wolman, 1984
Jemez	Jemez Canyon	2.8	12	fine sand	"
Arkansas	John Martin	0.9	30	fine sand	"
Missouri	Garrison	1.7	23	fine sand	"
Missouri	Fort Randall	2.6	23	fine sand	"
Missouri	Gavins Point	2.5	19	fine sand	"
Medicine Creek	Medicine Creek	0.6	3	fine sand	"
Des Moines	Red Rock	1.9	9	fine sand	"
Smoky Hill	Kanopolis	1.5	23	fine sand	"
Republican	Milford	0.9	7	fine sand	"
Wolf Creek	Fort Supply	3.4	24	fine sand	"
North Canadian	Canton	3.0	28	fine sand	"
Canadian	Enfaula	5.1	6	fine sand	"
Neches	Town Bluff	0.9	14	fine sand	"
Chattahoochee	Buford	2.6	15	fine sand	"
N. Canadian	Ford Supply	2.0	7	sand	Coldwell, 1947
Murab	Tashkeprinski	2.8	15	sand	Altunin, 1958
Syr-Daria	Farhadski	1.3	7	sand	"
Murab	Gindokushki	4.05	60.	sand	"
	Tedjenski	5.60	7	sand	"
Isar	Sylvenstein	0.90	24	sand, gravel	Weiss, 1984
Lech	Forgensee	0.60	10	sand, gravel	"
Saalach	Reichenhall	3.10	21	sand, gravel	"
Saalach	Reichenhall	4.60	47	sand, gravel	"
Inn	Iettenbach	2.50	54	sand, gravel	"
Inn	Neuötting	4.50	20	sand, gravel	"
Isar	Dingolfing	2.8	14	sand, gravel	"
Iller	Unterbalzheim	1.0	6	sand	"
Wertach	Schwabmünchen	1.8	5	sand	"
Danube	Faimingen	1.0	12	sand, gravel	"
Danube	Ingolstadt	1.8	14	sand, gravel	"
Isar	Niedereibach	2.0	4	d_{mean}=15.7mm	Tippner, 1973
Inn	Egelfing	1.0	7	d_{mean}=19.2mm	"
Rhine	Gerstheim	2.5	15 months	d_{mean}=25mm	"

* USBR - U.S. Bureau of Reclamation

** NHC - Northwest Hydraulic Consultants Ltd.

No doubt, due to the limited nature of the data, the values in the table may not be representative. Nevertheless they do emphasize the need to account for maximum possible scour depths when designing river engineering structures. Maximum degradation of uniform material is usually observed near the dam. Local variability in bed material size, and hence erodibility, can result in maximum degradation at a considerable distance downstream from the dam (table 2.9). Often the point of maximum degradation varies over time as illustrated by observations on the Colorado and Missouri rivers.

TABLE 2.9 Degradation downstream from dams: distance affected, downstream rate of migration and location of point of maximum degradation (Williams and Wolman, 1984, Altunin, 1958)

Years after dam closure	Distance downstream from dam affected by erosion (km)	Downstream rate of migration of erosion (km/year)	Distance from the dam where maximum degradation is observed (km)
	<u>Colorado River, Glen Canyon Dam</u>		
3	25	8	4
7	-		4
9	25		16
19	25		16
	<u>Colorado River, Hoover Dam</u>		
0.5	21	42	8
1	28	14	3
2	50	22	3
3	85	35	3
5	120	-	3
7	120	-	15
12	120	-	13
	<u>Arkansas River, John Martin Dam</u>		
9	26	3	22
24	26	0	4
30	26	0	22
	<u>Missouri River, Fort Peck Dam</u>		
13	75	6	17
18	75	-	17
23	75	-	23
36	75	-	17

Missouri River, Garrison Dam

1	12	12	28
7	19	1.2	6
11	18	-0.25	6
17	21	0.5	6
23	21	0	6

Saalach River, Reichenhall Dam

21	9	0.5	-
47	18	0.5	-

Murgab River, Tedjenskii Dam

4	42	10.5	-
7	60	6.0	-

Missouri River, Fort Randall Dam

2	5	2.5	11
5	13	2.3	1.6
8	14	0.3	11
15	14	0	11

Missouri River, Gavins Point Dam

5	15	3	2
10	14	-0.2	4
15	23	1.8	2
19	23	0	2

Des Moines River, Red Rock Dam

9	20	2.2	12

Wolf Creek River, Fort Supply Dam

7	7	1	0.3
19	7	-	0.3
27	7	-	0.3

North Canadian River, Canton Dam

1	7	-	1.8
11	7	-	1.8
18	7	-	1.8

Canadian River, Enfaula Dam

6	16	2.7	0.8
14	29	1.6	0.8

Red River, Denison Dam

3	7	2.3	15
6	27	-	15
16	27	-	1
27	27	-	15

Chattahoochee River, Buford Dam

7	7	1	2
9	9	1	2
12	-	-	2
15	10	0.2	2

2.2.3.5 Decrease in water levels Bed erosion, and associated changes in channel width downstream from dams, cause spatial and temporal changes in water levels (Fig. 2.32).

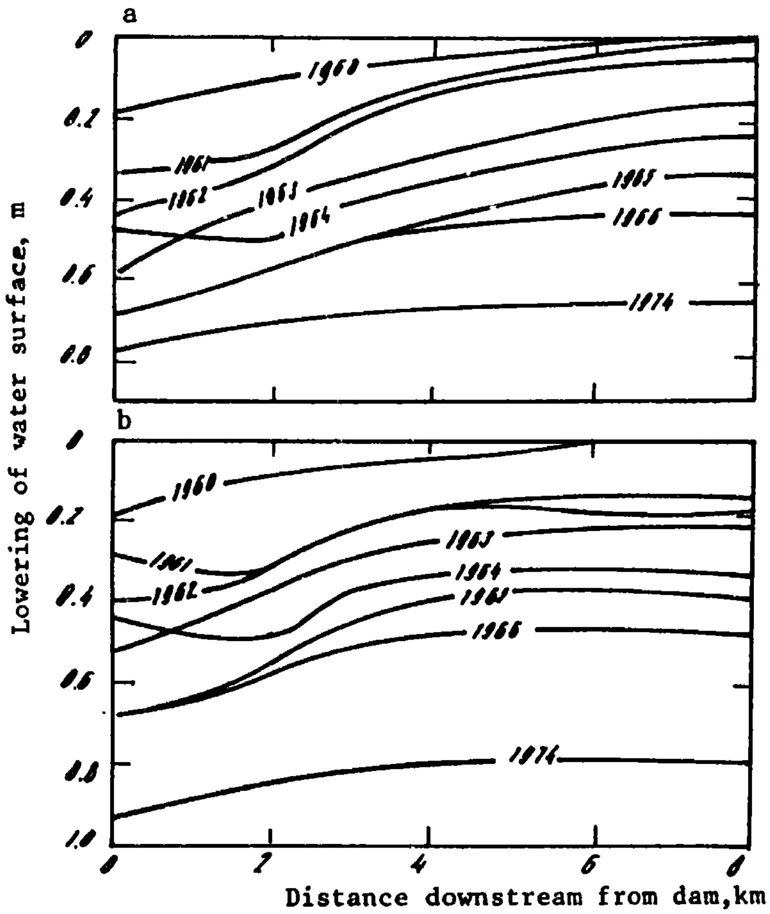


Fig.2.32 Lowering of the water surface downstream from Kanauski dam, river Neman, for the period 1960-1974 at discharges of a) 90m^3/s and b) 293m^3/s (Veksler and Donenberg, 1983)

Stage-discharge curves for the Ob river downstream from Novosybirski dam are illustrative of the changes that can occur at a particular site as a result of degradation (Fig. 2.33). Temporal changes in water levels are best obtained by observing changes in level for a specific discharge. The long term mean annual discharge is most appropriate for this purpose.

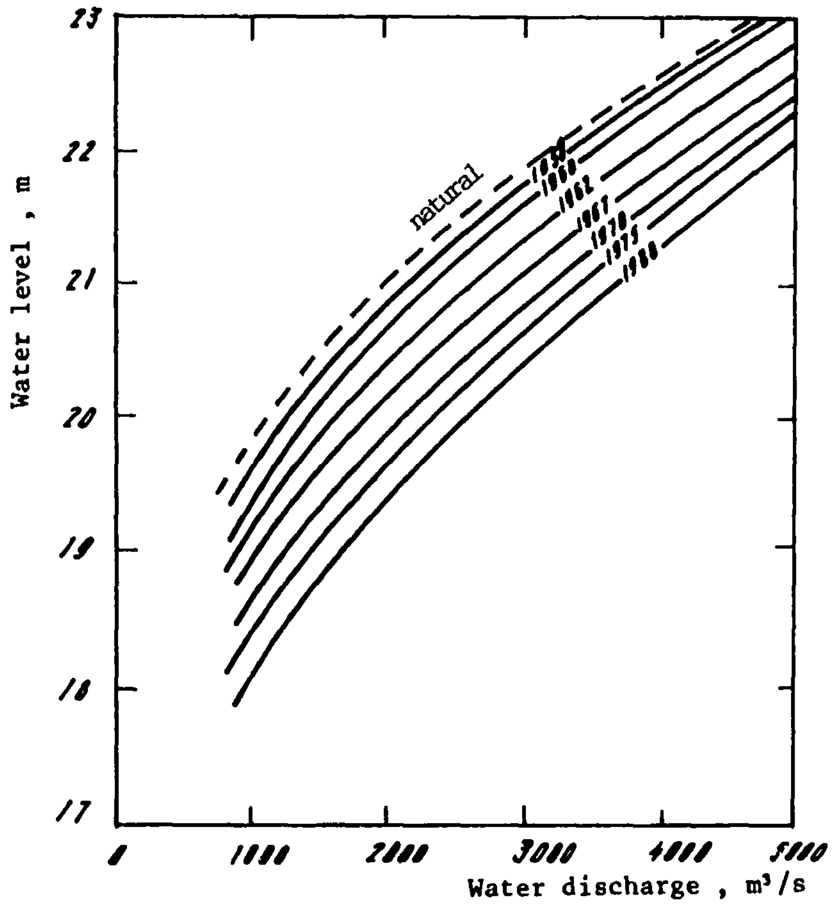


Fig.2.33. Stage/discharge curve; Ob river downstream from Novosibirski dam for the period 1958-1980 (Veksler and Donenberg, 1983)

Table 2.10 presents data for changes in water level corresponding to the long term mean annual discharge downstream from a number of dams. Graphical representation of these data, given in fig. 2.34, show that in most cases there is a significant reduction in water levels in the first few years after dam closure. Subsequently the rate of change decreases and, in some cases, ceases. Often stability may be only temporary and a major flood may breach any armour layer that had developed and allow further degradation. For example, water levels on the river Don downstream from Zimlianski dam decreased 68cm in the first 5 years after dam construction and then stabilised for the next 5 years. Ten years after dam closure water levels were lowered further, decreasing by 120cm after 20 years.

TABLE 2.10 Change of water level (cm) corresponding to a given long term mean annual discharge downstream from dams (after Veksler and Donenberg, 1983, Williams and Wolman, 1984)

No.	River	Dam	Mean water discharge (m^3/s)	Observational period (years)	Years after dam closure									At the end of the observation period	
					0	1	2	3	5	7	10	15	20	natural	according to design
1.	Volga	Rybinski	1120	16							−52	−52		−52	
2.	Volga	Gorkovski	1690	15	+26	+26	+26	0	0	0	−16	−35		−35	−10[x]
3.	Volga	Kuibishevski	7620	12	0	+13	+38	+55	+46	+35	+19			+10	−15
4.	Volga	Saratovskii	7830	4	−39	−37	−19	−2						+4	−15[xx]
5.	Volga	Volgogradskii	7960	11	0	0	−12	−19	−20	−24	−36			−50	−13
6.	Kama	Permski	1630	7	0	0	0	0	−20	−35				−35	
7.	Kama	Vodkinski	1710	10	0	−25	−25	−25	−25	−40	−50			−50	
8.	Ufa	Pavlovski	336	12	−30		−39				−47				
9.	Don	Tzimlianski	675	19	0	−24	−48	−68	−68	−68	−68	−96		−122	−40
10.	Dnepr	Kievski	1050	7	0	−34	−34	−40	−65	−65				−65	
11.	Dnepr	Dneproges	1650							+6		−44		−44	
12.	Daugava	Rijski	640	5		+43	+21	+9						+9	
13.	Neman	Kaunaski	293	12	0	−17	−28	−40	−68	−68	−68	−92		−104	−104
14.	Kura	Mingechauski	397	3	−80	−94	−107							−107	
15.	Kura	Varvarinski	397	10	−40	−58	−66	−88	−92	−92	−112			−112	
16.	Ob	Novosibirski	1640	9	0	−18	−29	−42	−53	−67	−82	−116		−156	−50[xxx]
17.	Irtysh	Ust-Kamenogorski	629	10						+20	+20			+ 20	−100
18.	Syr-Daria	Farhadskii		7				−72		−126				−126	
19.	Murgab	Tashkeprinski	50	15											
20.	Murgab	Gindukushki	55	60										−405	
21.	Murgab	Tedjenski	35	7						−560				−560	
22.	Colorado	Parker	90	5	+20	−90	−190	−170	−260					−260	
23.	Jemez	Jemez Canyon	0.37	5	−30	−50	−70	−100	−240					−240	
24.	Missouri	Fort Peck	85	40	0	0	−20	−25	−30	−60	−80	−120	−140	−150	
25.	Missouri	Fort Randall	464	17	0	−30	−30	−30	−30	−40	−50	−70		−70	
26.	Missouri	Gavins Point	312	20	−60	−65	−70	−70	−70	−75	−100	−120	−180	−180	
27.	Smoky Hill	Kanalopolis	0.50	20	0	0	−30	−30	−90	−90	−90	−100	−140	−140	
28.	Republican	Milford	3.4	10	+10	+10	0	−30	−50	−90	−130	−145		−150	
29.	N.Canadian	Canton	0.03	7	+20	+20	−40	−70	−100					−100	
30.	Red	Denison	3.7	18	0	−40	−50	−90	−100	−120	−120	−130		−140	
31.	Neches	Town Bluff	4.2	28	0	−10	−10	−10	−20	−40	−60	−70	−90	−90	
32.	Chattahoochel	Butford	12.2	15	0	0	−15	−20	−30	−40	−70	−100		−100	
33.	Rio Grange	Caballo	28.3	42	0	0	−10	−10	−15	−20	−50	−55	−60	−75	
34.	Marias	Tiber	2.8	20	−20	−22	−24	−24	−25	−20	−20	−20	−20	−20	
35.	Frenchman Creek	Enders	1.3	30	−20	−20	−20	−20	−20	−25	−25	−40	−50	−50	

x – 5 years after dam closure
xx – 15 years after dam closure
xxx – 50 years after dam closure

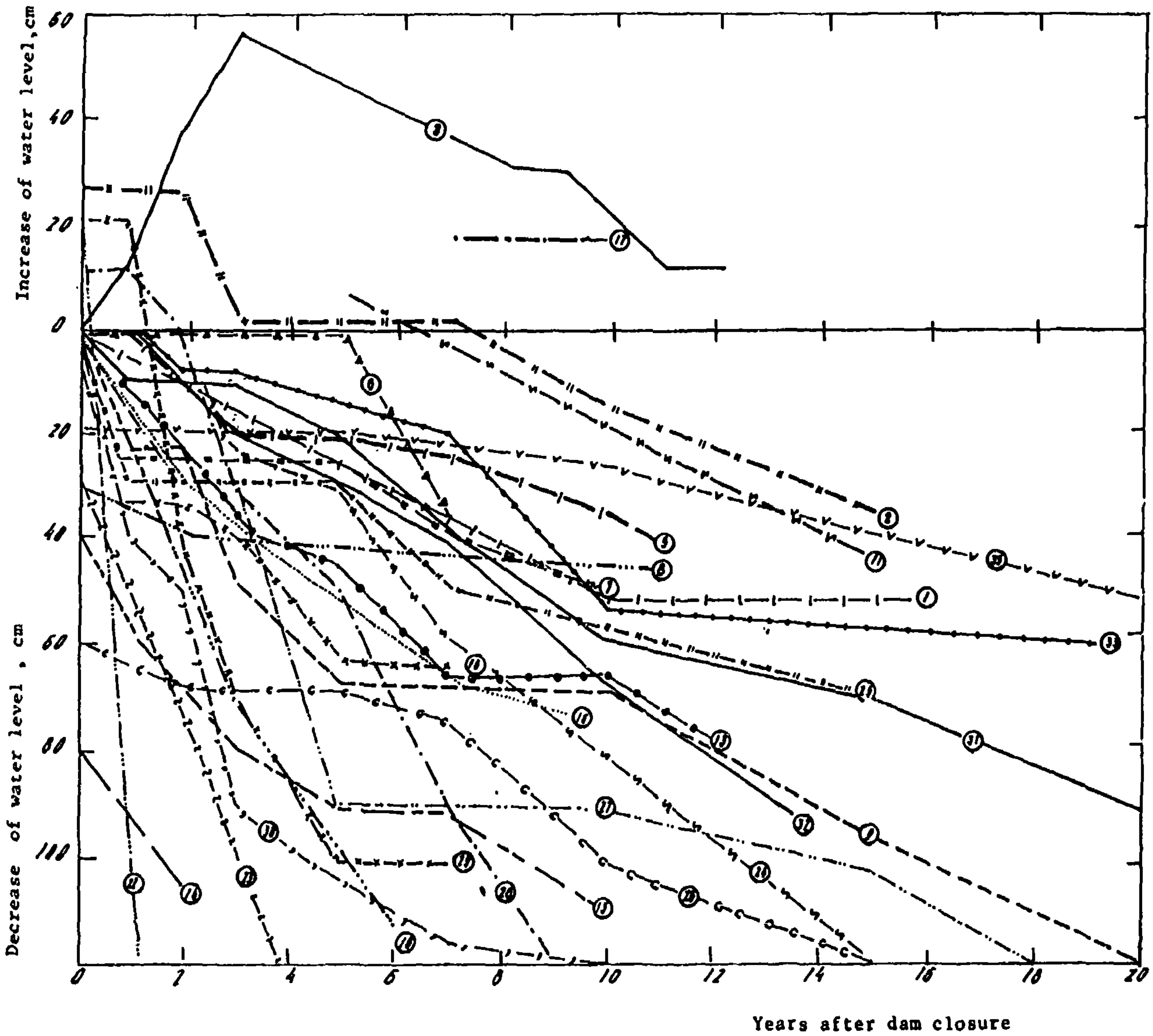


Fig.2.34. Change in water level corresponding to the long term mean annual discharge downstream from dams (numeration according to table 2.10)

Some rivers show a temporary increase in water levels, as for example downstream from Kuibishevski, Gorkovski and Canton dams, due to deposition of scoured material.

2.2.3.6 Length of degraded reach The length of degraded reach downstream from a dam depends on the local differences between the sediment load and the transport capacity of the river.

If non-erodible layers are exposed, or the bed becomes armoured by coarse material immediately downstream from the dam, then erosion will occur further downstream where transport capacity exceeds sediment supply. One example where this occurs is on the river Sar Daria below Farhadski dam (Fig. 2.25).

The length of reach subject to erosion varies between rivers and for a particular river, over time. Consequently the data presented in table 2.11 are simply illustrative of the length of channel affected by erosion for the time period indicated. In some cases, the length of degraded reach is limited by the existence of natural or artificial base levels, for example, confluence with the sea, lake, reservoir, series of reservoirs, or a weir.

Williams and Wolman (1984) showed that the length of the degraded reach increases with time (table 2.9). On the Colorado below Hoover Dam, the length of degraded reach 6 months after dam closure was 21km, 1 year after - 28km, 2 years after - 50km, 3 years after - 85km, 5 years after - over 120km. Similarly, on the river Saalach downstream from Reichenhall Dam the length of degraded reach was 9km 21 years after dam closure and 18km (Lane, 1955) after 47 years (Weiss, 1984) without stability being achieved.

Determination of the rate of migration of erosion downstream is of considerable theoretical and practical interest. Unfortunately data are very limited. The maximum recorded downstream rate of movement is 42km/year. Mean rates of migration can also be large; as high as 40km/year downstream from Iron Gate I dam on the river Danube and more than 30km/year downstream from Aswan dam.

2.2.3.7 Variation of bed-material grain sizes Bed-material size in the reach downstream from the dam exerts considerable influence on the nature, degree, rate and extent of degradation.

Bed-material is usually non-uniform, not only at a given site, surface and subsurface, but also along a length of river. Although there are coarse fractions in the bed material, these can normally be transported by the river under natural conditions. Decrease in peak discharges after dam closure, due to flood storage, reduces the ability of the river to transport the coarsest fractions of the bed-material. However, reduced sediment loads downstream from the dam allows the regulated flows to erode fine material from the bed and banks. This leads to a progressive coarsening of the bed material and the development of a surface armour layer. This surface armour protects the subsurface layers from erosion.

If the surface armour is breached during a major flood event, then erosion can be re-established until a new armour layer develops. This occurred on the Izar river downstream from Dungolfing Dam (Bauer and Burz, 1968) where the river bed had been stable for 4 years due to bed armouring, and became unstable after the passage of a large flood breached the armour layer.

Progressive downstream movement of the zone of erosion enables the armouring process to develop further downstream. Coarsening of the bed material has occurred downstream from Iron Gate I Dam on the Danube in the 8 years following dam closure. Over a long length of river the mean diameter of bed material was ten times greater than its value prior to dam construction (Raynov et al., 1979) (fig. 2.35). It is anticipated that the coarsening process will progress further downstream since hydraulic conditions are similar.

TABLE 2.11 Length of river affected by degradation downstream from dams

River	Dam	Length (km)	After dam closure (years)	Bed material	Source of data
S. Canadian	Conchos	30	10	sand, gravel	Hathaway, 1948
Middle Loup	Milburn	8	16	sand	USBR, 1945
Colorado	Hoover	120	13	sand, gravel	USBR, 1945
Colorado	Davis	52	30	sand, gravel	USBR, 1945
Sakarya	Sariyar Baraji	300	-	-	Simons & Senturk, 1977
Yellow	Sanmexia	68	4	fine sand	Li et al., 1980
Syr-Daria	Farhadski	250	7	sand	Altunin, 1958
Murgab	Tashkeprinski	190	15	sand	Altunin, 1958
Murgab	Tedjenskoe	60	7	sand	Altunin, 1958
Inn	Iettenbach	28	54	sand, gravel	Weiss, 1984
Isar	Dingolting	46	14	sand, gravel	Weiss, 1984
Danube	Iron Gate I	300	10	sand	Pechinov, 1984
Nile	High Aswan	551	18	sand	Shalash, 1982
Saalach	Reichenhall	47	47	sand, gravel	Williams & Wolman, 1984
Missouri	Fort Peck	75	13	sand, gravel	"
Isar	Niederahbach	9	4	graval	Tippner, 1973
Inn	Eggelfing	15	7	gravel	Tippner, 1973
Danube	Befinger Halde	36	8	gravel	Tippner, 1973
Rhine	Gerstheim	2.5	15 months	gravel	Tippner, 1973

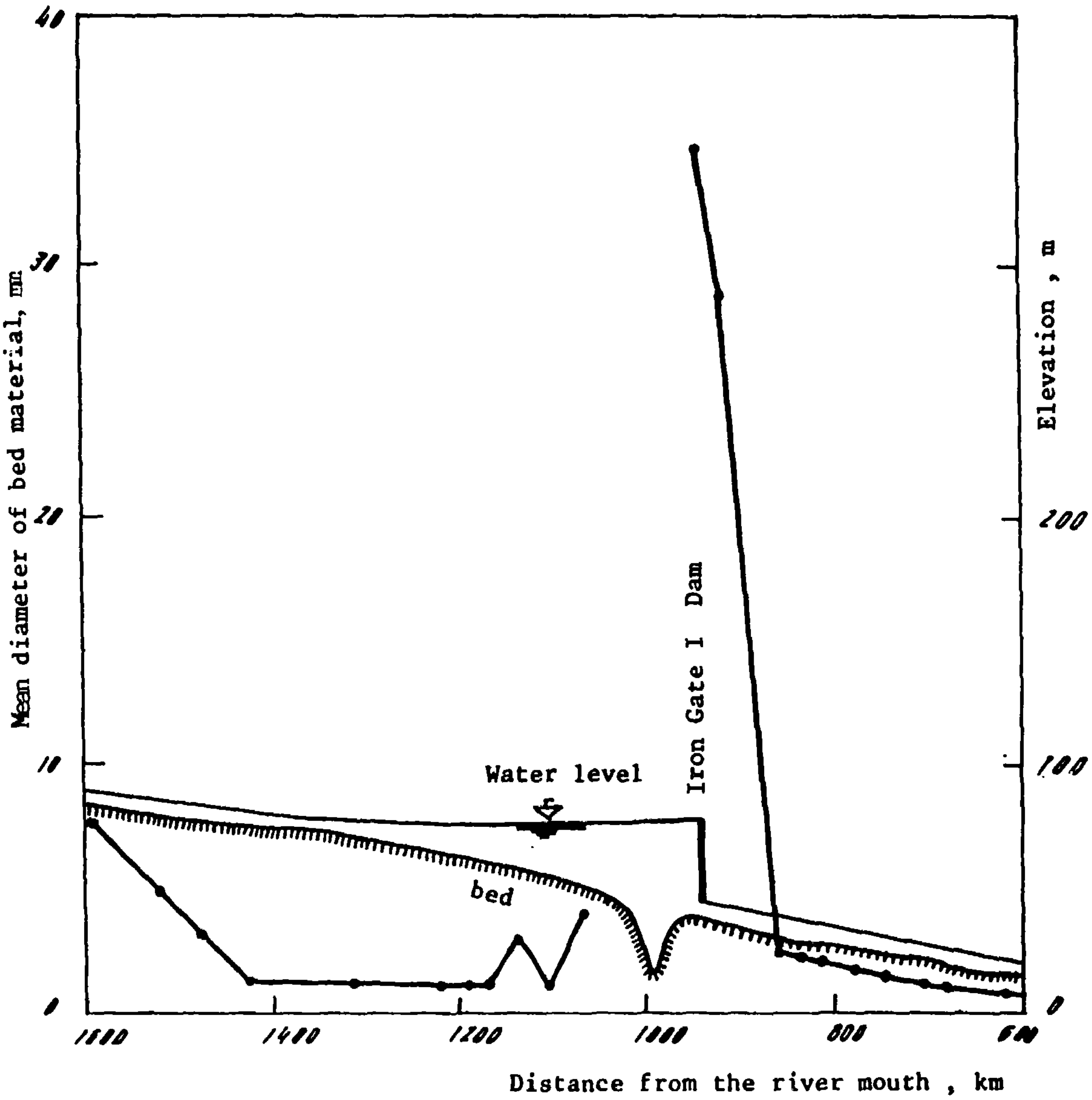


Fig. 2.35 Downstream change in bed material size and bed elevation on the Danube 8 years after closure of Iron Gate I dam (after Raynov et al., 1979)

Some examples of the change in the size of bed material downstream from dams have been obtained by Williams and Wolman (1984). In general there is a progressive coarsening of the surface bed material both in time and space, although they vary in detail due to specific site conditions.

A direct relation has been observed between mean bed-material size and the depth of bed degradation downstream from Kanopolis Dam (13 years after dam closure) and Denison Dam (16 years after dam closure) (fig. 2.36) probably because the mean bed material size does not vary considerably with distance and that subsurface variations are comparable between sites.

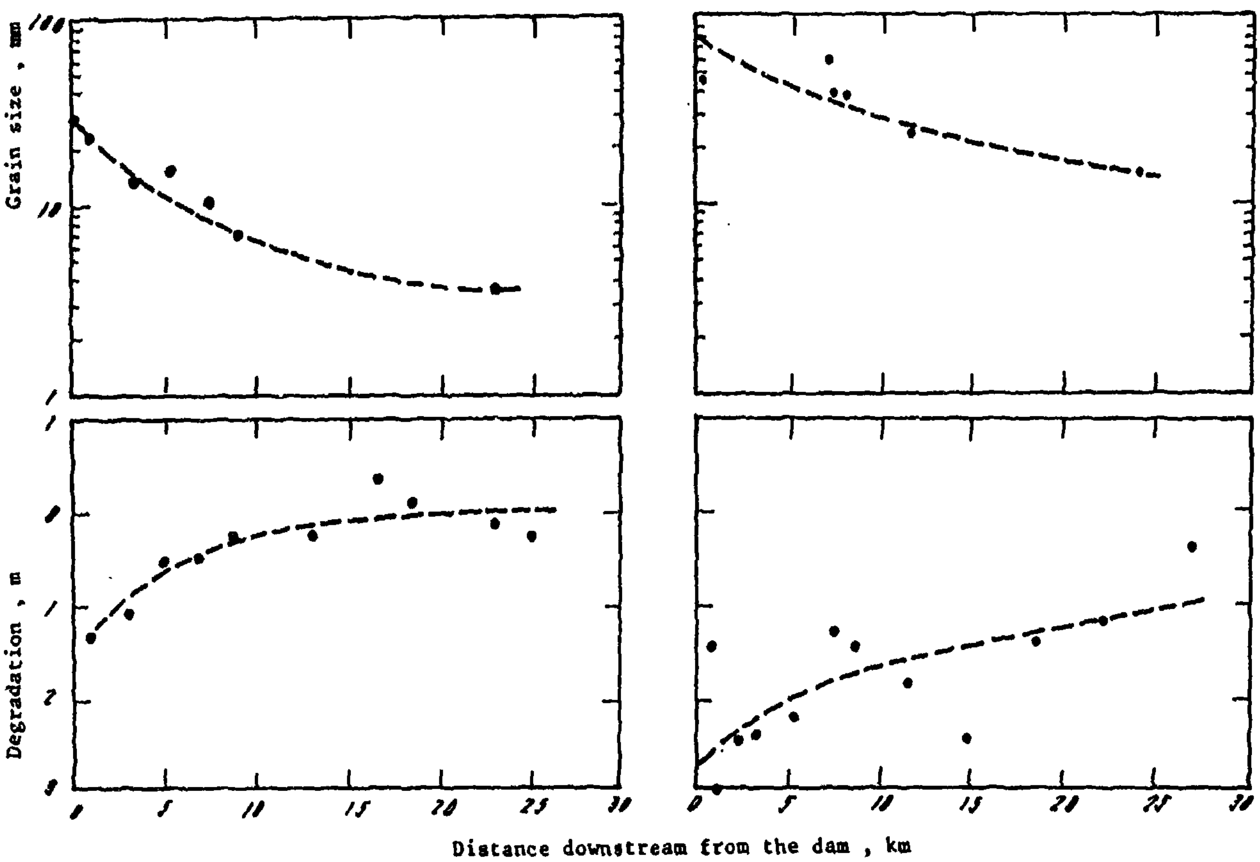


Fig. 2.36 Downstream changes in bed material size and bed degradation a) Kanopolis Dam (13 years after dam closure); b) Denison Dam (16 years after dam closure) (after Williams and Wolman, 1984)

A gradual increase in bed-material size, both in time and space, has been observed on the Colorado downstream from Hoover Dam (fig. 2.37). During the first year after dam closure the median bed material size 20km and 70km downstream from the dam was 0.15-0.20mm. After 3 years, the median bed material size had increased to 23mm just downstream from the dam, while only a small increase was observed between 10-40km and no change at 70km. Thirteen years after dam closure the median bed material size below the dam and 40km downstream had increased to 80mm and coarsening had occurred at 135km.

Bed-material grain sizes along the river Colorado were practically uniform throughout the entire 130km river reach downstream from Park Dam (median size approximately 0.15mm) (fig. 2.38) at dam closure. Five years later the median diameter at distances 26 and 64km below the dam increased to 0.4mm, while at 130km it remained the same. After 15 years the median grain size 26km downstream from the dam increased to 20mm, i.e. over 130 times larger than the median grain size under natural conditions. The maximum value of the median grain size 64km from the dam was observed 20 years after dam closure, while at 130km it tends to increase slightly throughout the entire 35 year period.

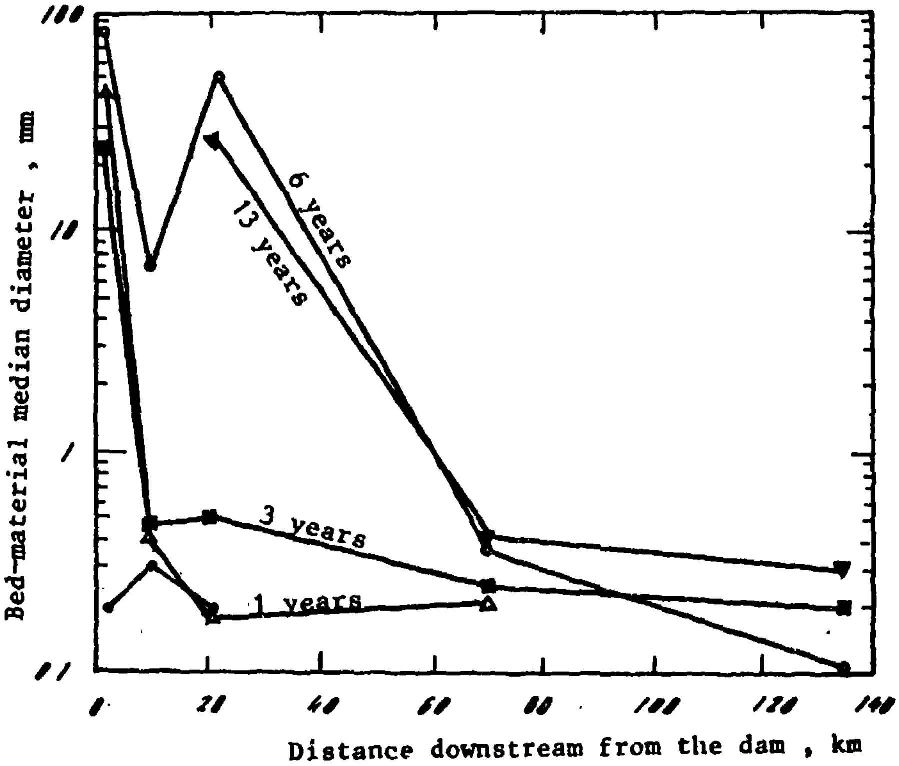


Fig. 2.37 Changes in median bed material size, downstream from Hoover dam, river Colorado, 1, 3, 6 and 13 years after dam closure (Williams and Wolman, 1984)

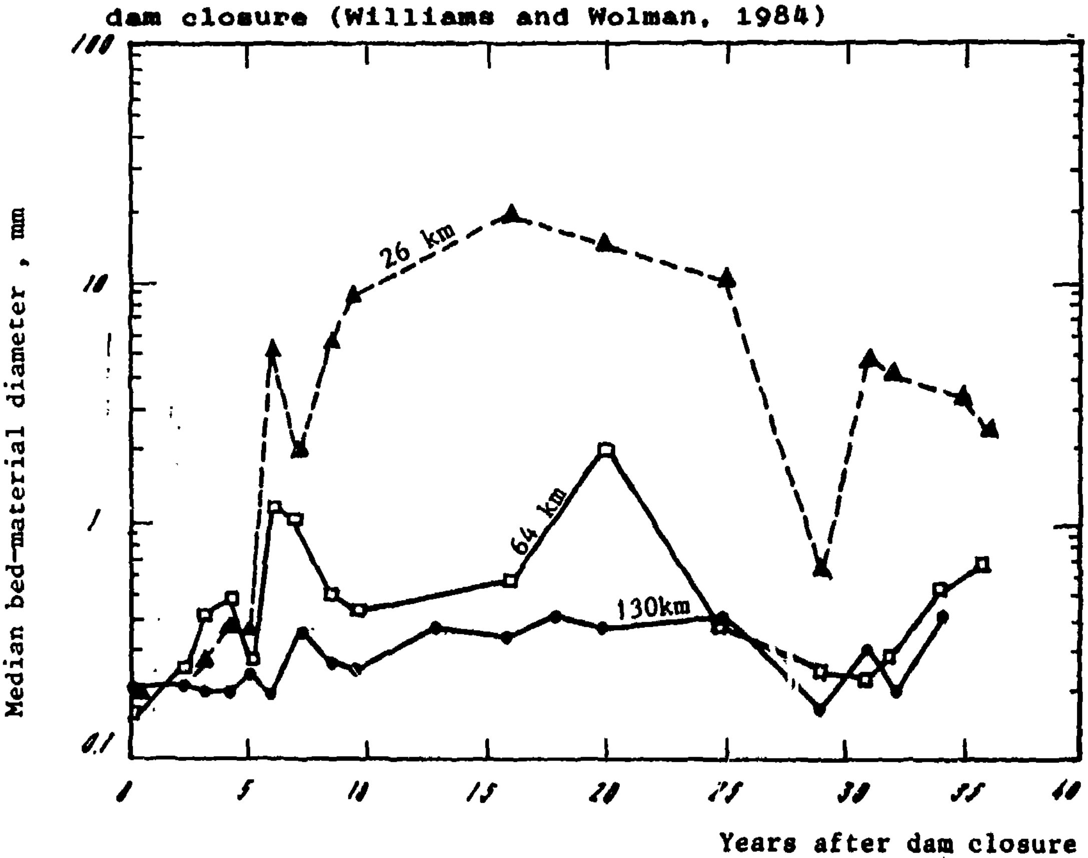


Fig. 2.38 Changes in bed material size 26, 64 and 130km downstream from Parker Dam, river Colorado after dam closure (Williams and Wolman, 1984)

Thus, the rate of bed degradation is retarded by armouring and, under certain conditions, prevents further erosion over long stretches of river. This means that within the armoured reach sediment transport rates are progressively reduced with the result that bank erosion occurs, or bed degradation takes place further downstream. Therefore, the degree and extent of degradation has a major influence on the bed material grain size distribution.

Although there are a number of methods available for predicting the development of an armoured layer, unfortunately they give different results indicating the need for further research on this topic.

2.2.3.8 Stabilization of the river bed Determination of the conditions controlling the cessation of erosion is of considerable theoretical and practical interest.

Downstream from dams, stabilization of the river bed is possible by:

1. Increasing sediment load up to a value corresponding to the transport capacity of the flow;
2. Decreasing the transport capacity of the flow;
3. Armouring of the river bed by the coarse component of the bed material.

Sediment discharge downstream from a dam will increase after sedimentation has partially or completely filled the reservoir. The time for this to occur depends on the sediment yield of the catchment, the size of the reservoir and its trap efficiency. In the case of reservoirs with a small regulating capability, this is relatively quickly achieved. For instance, 3 years after the Etenbach reservoir on the river Inn was commissioned, sediment discharged from the reservoir (fig. 2.3) enabled the original longitudinal profile of the river to be restored. In contrast the increase in sediment discharge downstream from reservoirs which have a large regulating capability and trap efficiency is particularly slow. Thus, the High Aswan reservoir on the Nile transmits only 1-2% of the total sediment discharge in the first years after dam closure (fig. 2.2), while after 500 years it is predicted that it will transmit 8% of the sediment supplied to the reservoir.

The sediment discharge downstream from dams is also influenced by contributions from tributaries. Furthermore, a reduction in water levels, due to bed erosion and run-off regulation during flood events, inevitably leads to a temporary increase in sediment yield from the tributaries as a result of erosion in their lower reaches.

Many interesting solutions have been proposed to increase sediment run-off to compensate for reduced loads below the dam in order to stop erosion. For example, on the Rhine, it was predicted that the river bed would be lowered by 15m unless material was injected into the river. To stablize the bed, 170,000m^3/year of coarse material is tipped into the river over a 50km reach, 334-390km downstream from the dam, and just below a major set of locks (fig. 2.39) (Untersuchungen, 1981).

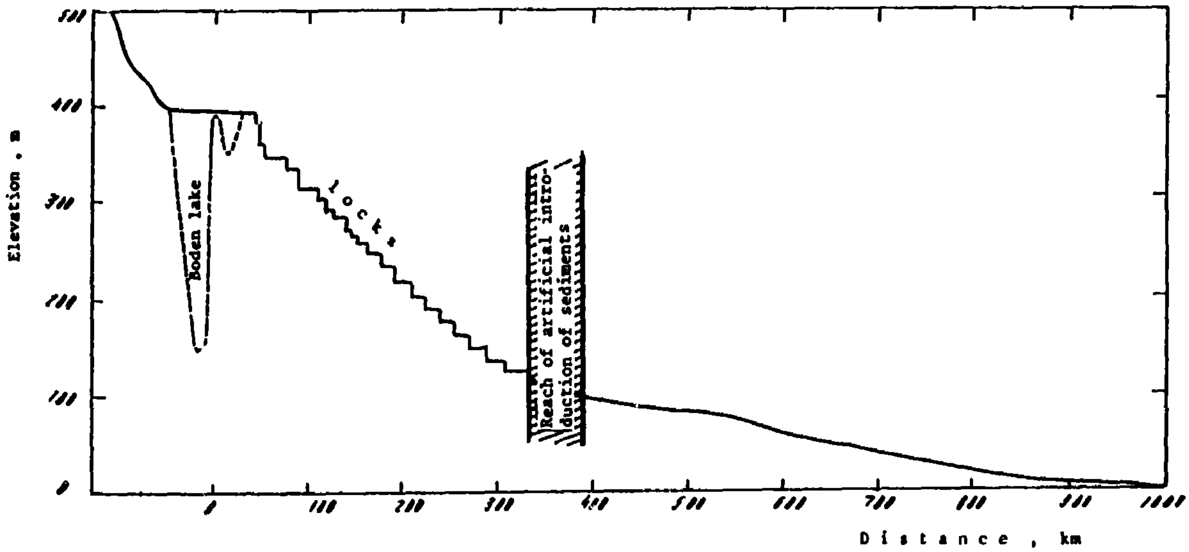


Fig. 2.39 Longitudinal profile of the river Rhine (after Untersuchungen..., 1981)

The gradual decrease in the transport capacity of the river downstream from dams, as a result of erosion, is self stabilizing. The increase in cross sectional area of the river, reduced slope, and related decrease in flow velocity are the dominant factors responsible for reducing the transport capacity. The rate of which this effect develops depends mainly on the erosional resistance of the bed and bank material and the degree of bed armouring.

Engineering works aimed at decreasing channel slope downstream from dams, by building weirs and barrages, reduces erosion upstream from the check dams but not downstream from them. These engineering structures reduce the transport capacity in the backwater zone which reduces sediment transport rates. Therefore the erosion process is transmitted further downstream.

A typical example of this process has been observed on the Danube following the construction of Iron gate I dam in 1970 (Fig.2.40). In the first ten years following dam construction degradation affecting 300km of channel. The most serious erosion being observed just downstream from the dam. In 1984 Iron Gate II dam was built and this has a much lower regulating capability and trap efficiency. As this does not appreciably change the transport capacity of the river or the calibre of the transported material, it is likely to increase degradation downstream.

Armouring of the bed (formation of a protective surface layer) by the coarse component of the bed material downstream from a dam is often the factor controlling the development of a stable condition. Once formed erosional activity is transmitted downstream where the process is repeated.

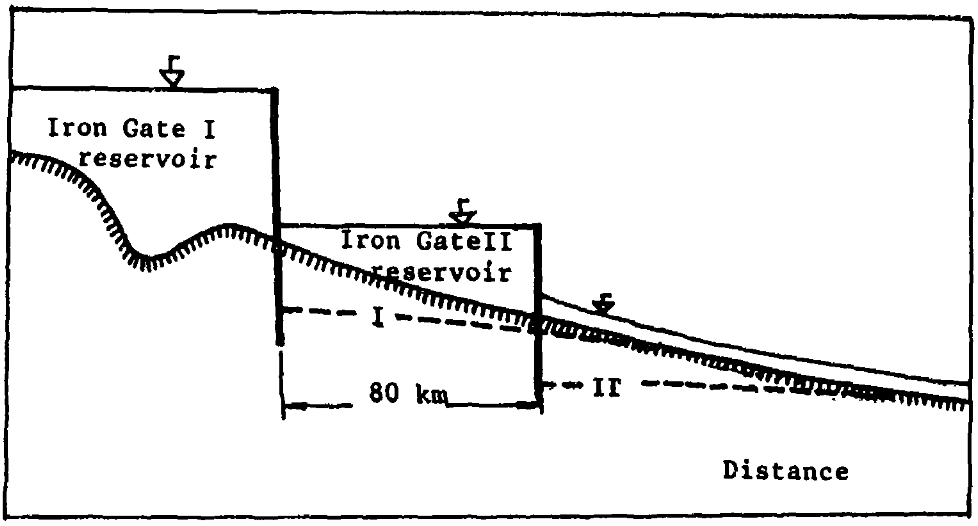


Fig.2.40 Degradation downstream from Iron Gate I dam on the river Danube

The development of an armour layer depends on the grain size of the bed material, particularly the presence of material with a threshold velocity for initiation of motion greater than maximum flood velocities during reservoir releases. In some cases erosion may reveal layers of non-erodible or slightly erodible material, as for example downstream from Farhadski dam on the river Sar Daria (Fig.2.25). Alternatively gravel may be artificially introduced into the river in order to stabilize some sections of the river bed as, for example, on the Missouri.

Due to the fact that in most rivers the bed material is heteorgeneous, both in depth and distance, the final stable profile may be quite irregular.

The multiplicity of factors that can influence erosional activity poses problems for predicting degradation downstream from dams.

Investigations into erosion on the Nile downstream from High Aswan Dam since its closure in 1966 illustrates the problems that can arise (Fig.2.41). The proponents of dam construction predicted that degradation of the bed would be negligible, while the opponents contended that the bed would degrade by 45m. Field measurements of actual degradation by Shalash (1983) proved that it was much smaller than predicted values (table 2.12). On the basis of hydrological observations 3 years after the closure of High Aswan Dam, Hammad (1982) concluded that degradation would be relatively small.

Predictions of large scale erosion on the Nile are based on the assumption that stabilization of the river bed would be achieved when a new slope had formed corresponding to a 98% reduction in sediment discharge. The potential for bed armouring was ignored. In contrast, Hammad considered that armouring of the bed surface was of primary importance. According to him armouring precludes the need for a significant reduction in slope.

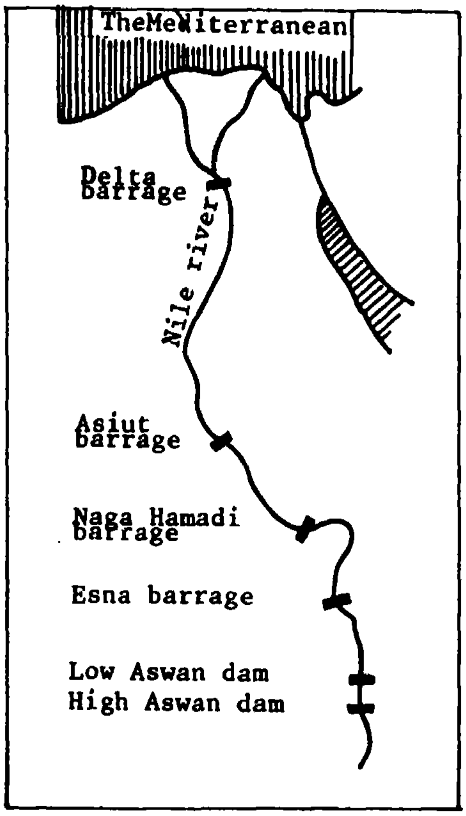


Fig. 2.41 Dams and barrages on the river Nile

TABLE 2.12 Reduction in water levels downstream from barrages and dams on the river Nile (after Shalash, 1983)

Dam	Distance downstream from the dam (km)	Height of dam (m)	Year of dam closure	Predicted reduction in water level by 1982: Mostafa 1957	VBB 1960	Hydro-project 1973	Shalash 1965	Observed reduction in water level by 1982 (m)
High Aswan Dam	0	100	1969	8.5	3.5	3.0	2.5	0.6
Low Aswan Dam	6	40	1902	-	4.0	-	-	0.9
Esna barrage	173	5	1908	9.0	3.5	3.5	2.5	0.8
Naga Hamadi barrage	366	5	1908	7.0	3.5	3.5	2.5	1.0
Assiut barrage	551	5	1902	6.5	3.5	3.0	2.5	0.7
Delta barrage	960	3.5	1936	-	-	-	-	-

Results to date indicate that Hammad's conclusions were correct. After about 1m of erosion an armour layer, 6cm thick, formed on the surface, increasing the roughness and stability of the river bed. It should be noted that such a process is possible on the Nile because flood storage precludes the release of high discharges which would destroy the armour layer and precipitate further erosion.

2.2.4 Changes in channel width

Considerable attention has been paid to the study of bed erosion and related water level changes downstream from dams, while little attention has been given to changes of channel width even though it may be significant.

Observations indicate that bed degradation downstream from dams, or rapid changes in water levels on rivers which are regulated for hydro-electric power production can cause serious bank erosion if they are not resistant to erosion. However, where much of the streamflow is diverted from the river, width can be reduced by deposition and vegetation encroachment.

Williams and Wolman (1984) in their study of changes in channel width downstream from dams observed, on the basis of comparatively limited data from the USA, that 22% showed no change in width, 46% an increase (Fort Peck, Gavins Point, Medicine Creek, Town Buff, Fort Randall), 26% a decrease (Jemez Canyon, John Martin, Fort Supply, Canton Dams), 5% first widening then narrowing (Canton Dam) and 1% narrowing then widening. The maximum increase of river width was observed to be 100%, while the maximum narrowing was 50%.

A large number of factors influence river width downstream from dams: extent of streamflow regulation, water distribution systems, diurnal flow fluctuations, especially with hydro-electric power stations, erosional resistance of the banks and the degree of bed erosion. Obviously changes in width decline downstream as the magnitude of the reduction in water and sediment discharge decreases. Nenov's studies (1985) showed that in most cases a reduction in channel width is observed and may be as high as 70% downstream from reservoirs in which a large part of the streamflow is diverted for irrigation or water supply. Any reduction in channel width reduces the flood capacity of the channel, which can negate the effect of the reservoir in reducing flooding.

Gradual and systematic changes in channel width and depth at a site 8km downstream from Gardiner Dam on the South Saskatchewan river 13 years after dam closure is shown in figure 2.42 (Galay, 1983). In constrast rapid changes in channel width are observed where the river is braided. The Yellow river downstream from Sanmexia Dam (Li, 1980), where degradation reduced water levels by 4m in 4 years (fig.2.43), is another example of this process. Erosion occurred over a distance of 68km and channel width was considerably decreased. Where bank erosion occurs material is supplied to the river which may, to a certain extent, make up for the reduction in the supply of material from upstream.

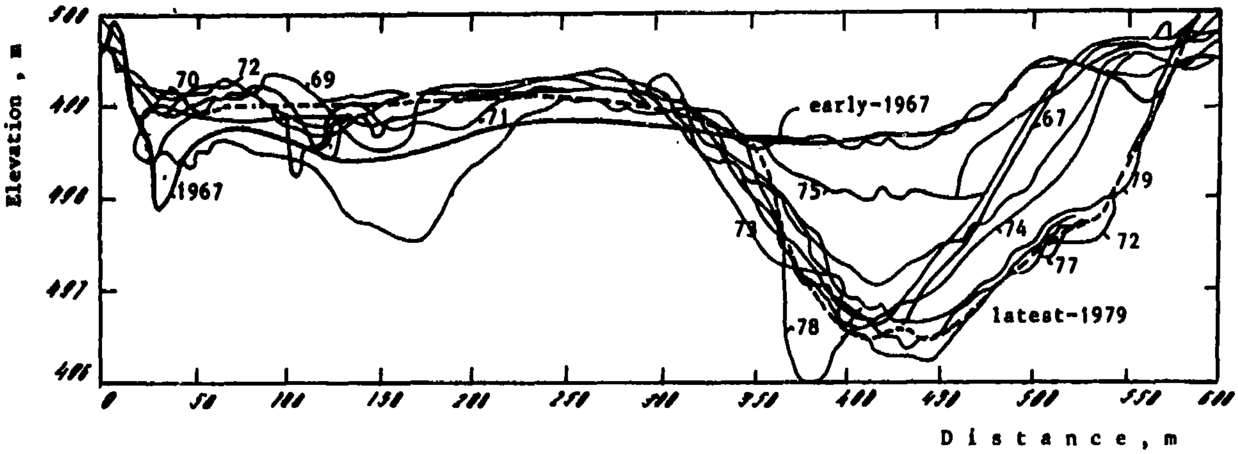


Fig.2.42 Changes in cross section in 13 years following dam closure at a site 1.6km below Gardiner dam, South Saskatchewan river (after Galay et al., 1983)

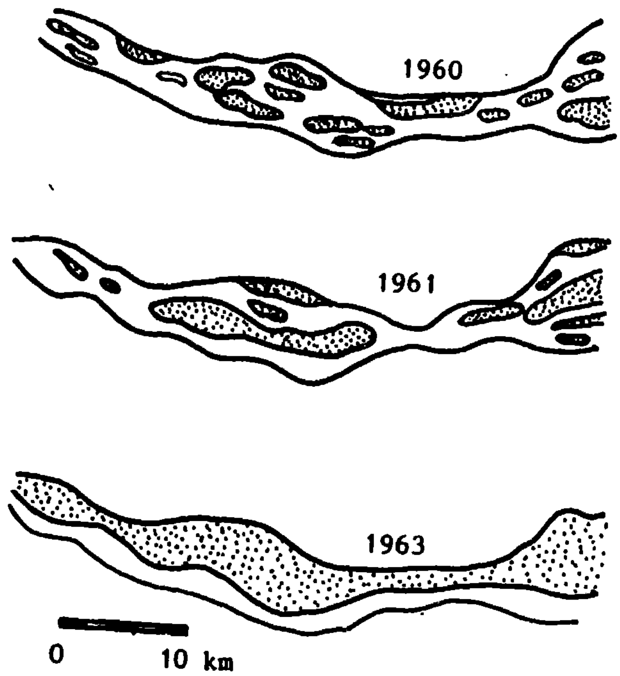


Fig.2.43 Changes in the plan geometry of the Yellow river downstream from Sanmexia Dam in 4 years after dam closure (after Li et al., 1980)

Occasionally river regulation and interbasin water transfers can cause degradation down to bedrock. This has occurred on Five Mile Creek, Wyoming, following a fifteen fold increase in discharge for an irrigation system. Subsequent bank erosion increased channel width to 10m in the rock chute sections and 300m in sections with alluvial banks (Lane, 1955).

Williams and Wolman (1984) noted that downstream from Fort Peck dam about 60-70% of the eroded material come from the banks and 30-40% from the bed, while on the Red river and North Canadian river 50-95% of the eroded material came from the banks. In contrast, on some reaches of the river Colorado downstream from dams, channel width is stable and the river bed erodes. Some typical changes in channel width, for the period from dam closure until it stabilizes, are given in fig.2.44. Although width adjustment is very variable, stability is eventually achieved.

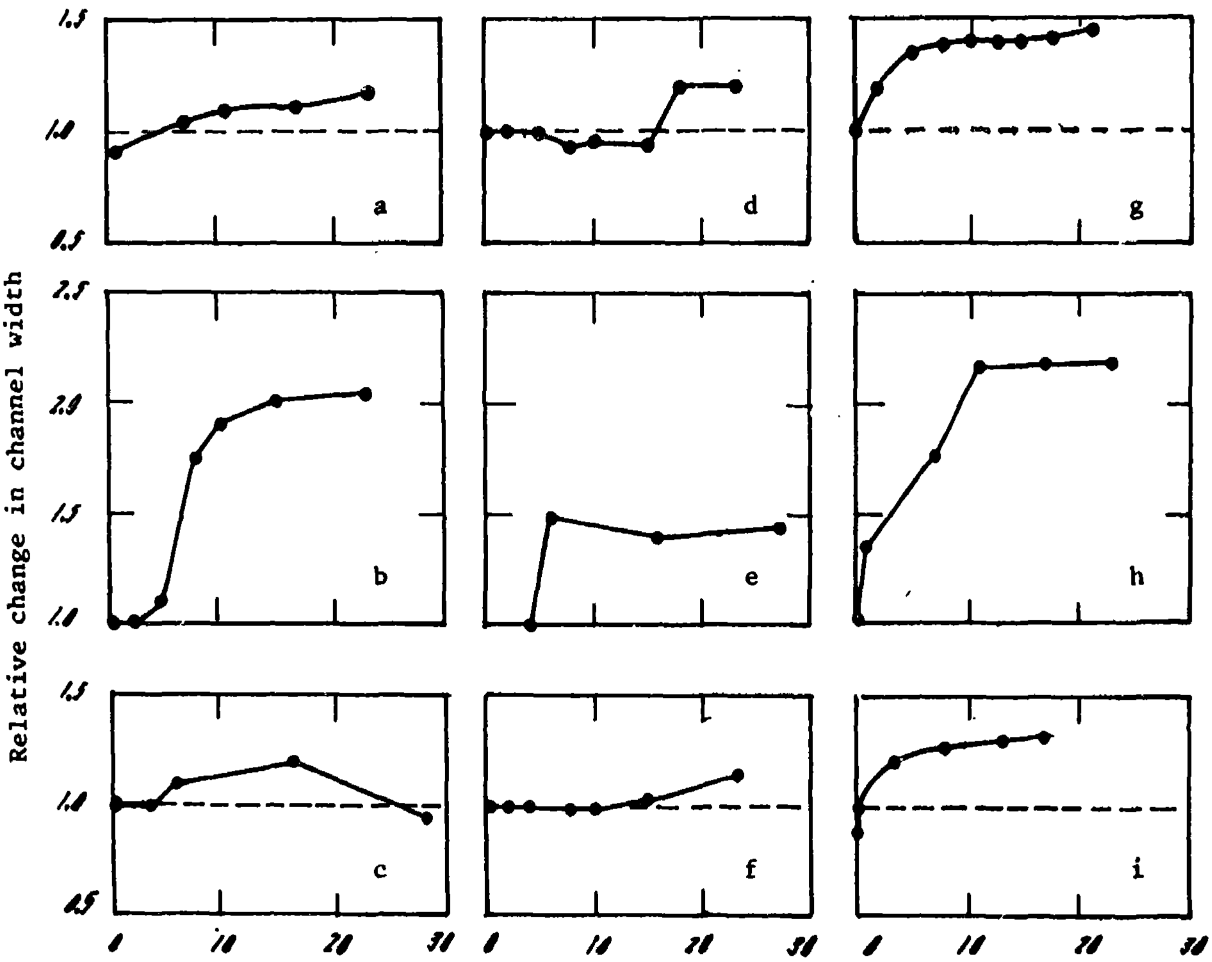


Fig.2.44 Examples of relative increase of channel width with time (after Williams and Wolman, 1984)

a. Missouri river, 61km downstream from Gavins dam
b. Missouri river, 19km downstream from Fort Randall dam
c. Red river, 150km downstream from Denison dam
d. Missouri river, 1.6km downstream from Fort Randall dam
f. Missouri river, 11km downstream from Fort Randall dam
e. Red river, 80km downstream from Denison dam
g. Missouri river, 14.5km downstream from Gavins Point dam
h. Missouri river, 47km downstream from Garrison dam
i. Missouri river, 48km downstream from Gavins Point dam

Vegetation plays an important role in limiting bank erosion due to their root systems increasing the tensile strength of the bank material. Reduction in the magnitude and frequency of floods downstream fron the dam encourages the development of shrubs and trees on the channel margin. These vegetated sections help to prevent bank erosion by decreasing the local flow velocity and encouraging deposition, regardless of the fact that immediately downstream from the dam a sediment deficit exists.

3. EROSION DUE TO REDUCTION IN BASE LEVELS

A reduction in base level, by the removal of rapids or dredging bed material, can cause degradation. Base level may be a lake, the sea, a reservoir, a main river or a local channel control. Lowering the water level leads to increased slope, velocity and transport capacity and, as a result, initiates erosion which gradually proceeds upstream (fig.3.1).

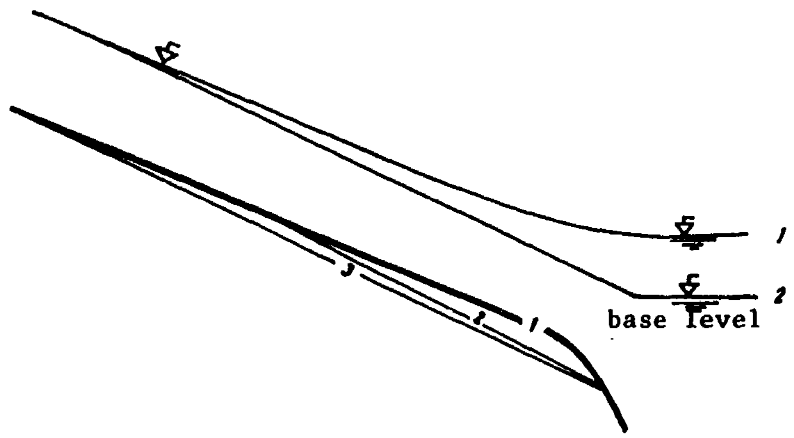


Fig.3.1 Change of water level and river bed profile due to a reduction in base level

Erosion of the bed a of main river often results in lowering the base level of its tributaries. Downstream from a dam water levels are also reduced during flood events which effectively rejuvenates the tributaries and precipitates erosion.

Galay (1983) gave the following interesting example illustrating this phenomenon. The I-29 interstate highway leading from Iowa into South Dakota crosses the Big Sioux river about 2.4km upstream of its confluence with the Missouri river. Towards the end of March 1962 a flood took place on the Big Sioux, while water levels in the Missouri were kept constant by controlling releases from several upstream reservoirs. The difference in water levels on the Big Sioux and the Missouri on 26 March was about 1m rising to 5.7m on 1 April. As a result serious degradation on the bed of the Big Sioux occurred which resulted in the collapse of a highway bridge.

As a result of tributry erosion, aggradation can occur downstream from its junction with the main river. This has occurred at the confluence of the river Samary with the Volga 100km downstream from Kuibishevski Dam. On the highway bridge over the Samary near its confluence with the Volga, flood flow velocities of 4-5m/s have been measured, while under natural conditions, pre-dam construction, they did not exceed 2.0-2.5m/s. Due to erosion on the Samary, sediment is being deposited in the Volga with the result that navigation depths are becoming limited (Veksler and Donenberg, 1983).

Regulation on the river Don by Tzimlianski Dam rejuventated the North Donetz during flood events. In 1953 the flood discharge on the North Donetz was very high (up to 3500m^3/s) while flows on the Don downstream from the dam did not exceed 60m^3/s. The reduction in base level and associated increase in the slope of the lower reach of the North Donetz (fig.3.2) caused substantial erosion of bed material which was subsequently deposited in the Don. In order to maintain navigation depths, considerable dredging has been carried out (Serebriakov, 1970).

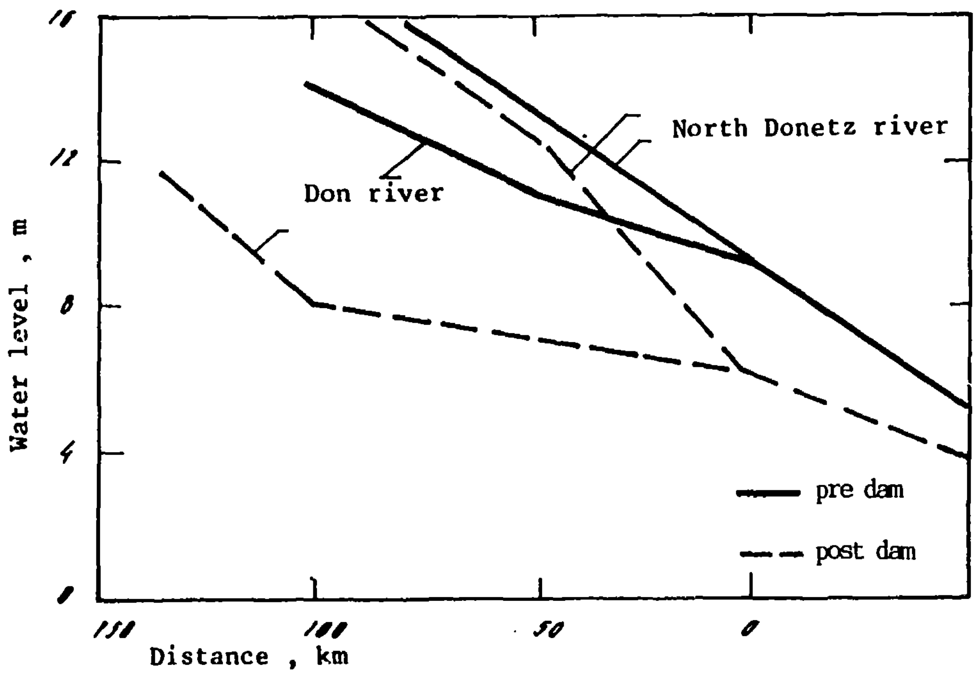


Fig.3.2 Water surface profiles of the Don river, downstream from Tzimlianski dam, and the North Donetz river (Serebriakov, 1970)

Changes in sea level can also adversely affect erosion rates. A 2.5m reduction in the level of the Caspian Sea (fig.3.3), partly due to natural processes but also due to increased irrigation in the river basins draining to the Caspian, has resulted in erosion.

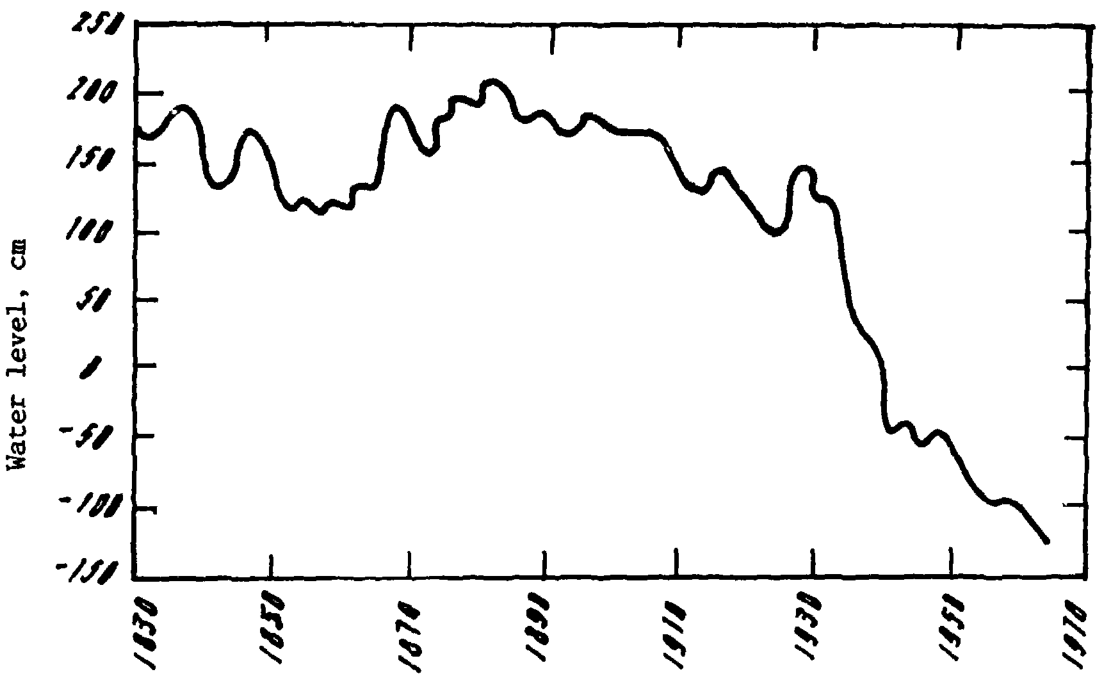


Fig.3.3 Relative changes in the level of the Caspian Sea for the period 1830-1965

Reservoir water levels, as they respresent the base level for inflowing rivers, have a major effect on reservoir sedimentation. Consideration should be given to this aspect when computing rates of sedimentation.

4. EFFECT OF DREDGING ON CHANNEL STABILITY

Dredging bed material not only results in a lowering of the river bed and water levels, it can also cause progressive degradation both upstream and downstream. Data show that an increasing amount of river alluvium is being used as building aggregate in a number of regions of the world. Removal of material from rivers alters the natural sediment transport regime, both upstream and downstream from the point of sediment removal, which can affect the stability of existing river engineering works. Predicting the degree and extent of erosion associated with planned dredging operations is of paramount importance in order to safeguard existing structure.

Usually dredged depths and widths are related to the size of the channel while the length will exceed the width of the river. The annual removal of material is usually 1-2 times the longterm mean annual bedload discharge.

Lowering of the bed and water levels due to dredging causes local changes in river slope over the dredged section and upstream and downstream from it (Fig. 4.1.).

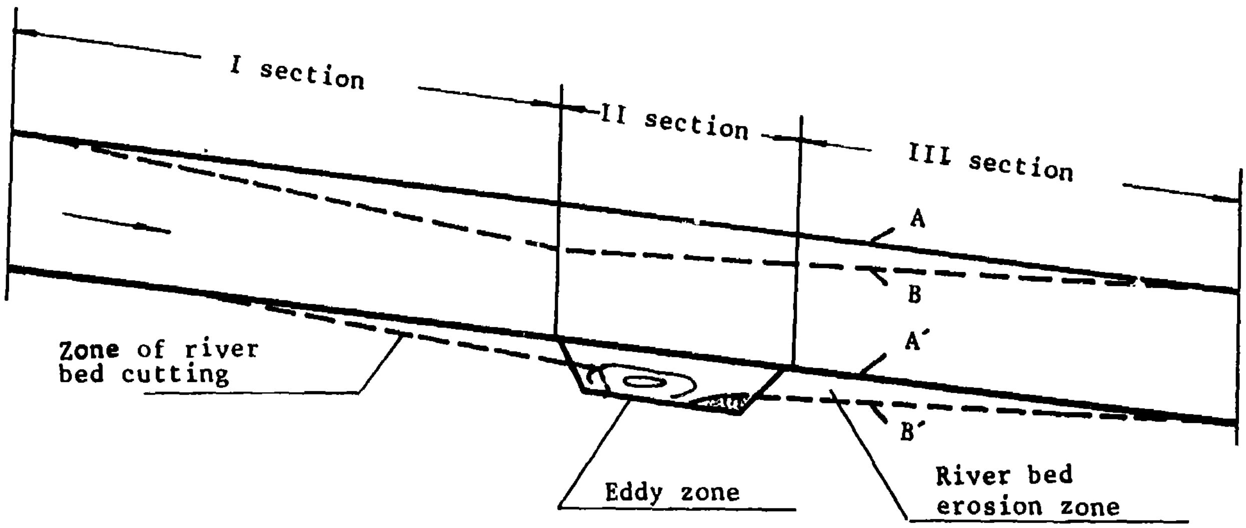


Fig.4.1 Changes in bed and water levels in dredged reach (Snishchenko et al., 1982)

AA' - water and bed surface before dredging

BB' - water and bed surface after dredging for a period of time

The increased water surface slope upstream results in an increased flow velocity. The latter may also arise from a reduction in surface resistance due to destruction of the coarse armour layer. Increased velocities encourage further erosion which results in a lowering of the water surface. Bed load and coarse suspended load derived from upstream are deposited in the dredged section.

As a result of bed load being trapped in the dredged reach, erosion occurs further downstream, with associated lowering of bed and water levels, since the transport capacity of the river has not changed. This is responsible for a further reduction in water levels upstream.

The reduction in water levels lowers groundwater levels in the flood plain. This can alter the soil water regime of the flood plain adjacent to the river, which can adversely affect trees (perennial vegetation) whose root systems cannot adapt quickly to rapid changes in water levels.

The length of channel affected by erosion depends on a variety of factors, including the amount of material dredged, bedload transport volumes, size of dredged section and calibre of the bed load. Observations show that it can exceed 20-30 times the width of the river bed. Removal of 400,000 m^3/year of bed material from the river Maritza (Bulgaria) downstream from Plovdiv caused the river to degrade by 2.5m. Erosion subsequently spread downstream and upstream (Fig. 4.2) endangering the foundations of the city bridges and bank support walls. One bridge, built over 120 years ago, collapsed.

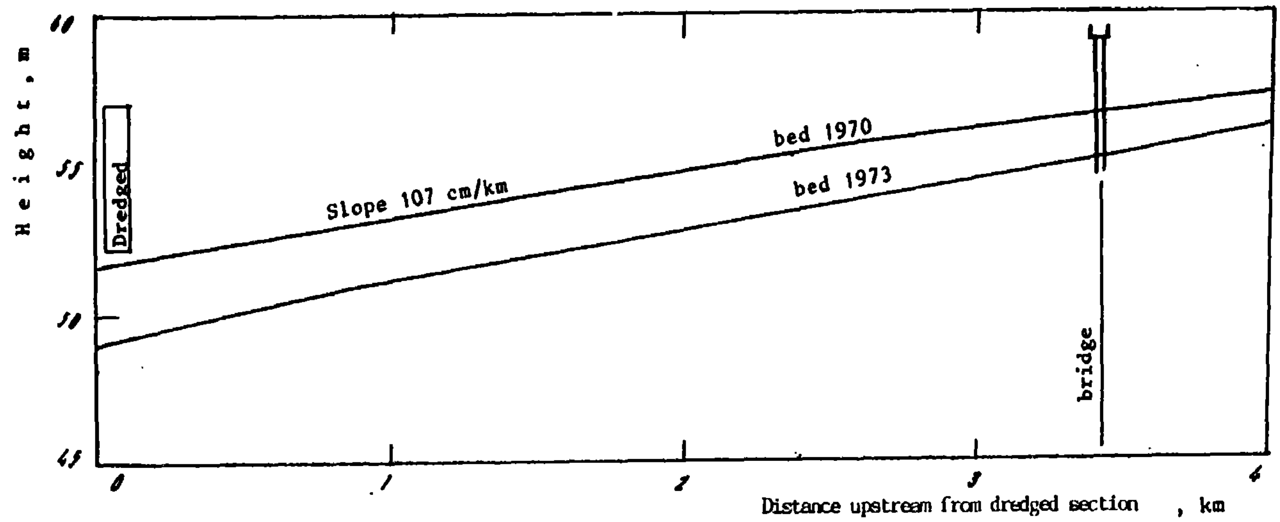


Fig.4.2 Degradation on the river Maritza, Plovdiv, (Bulgaria) as a result of dredging

An example of the effect of removal of armour material from the bed of the Chao Phraya is presented by Tingsanchali and Overbeek (1980). Dredging at a point 50km below Chao Phraya dam caused headward degradation towards the dam (Fig. 4.3). If dredging continues erosion may endanger the foundations of the dam. Prior to dredging activity, degradation had not occurred downstream from the dam because of the reservoir's small regulating capacity.

Another example of the adverse effect of dredging is presented by Agostini et al. (1981). On the river Brenta in Italy dredging has been carried out at a site just downstream from an autostrada bridge (Fig. 4.4). In order to stop headward erosion and provide stability for the bridge a weir was constructed. As a result of incorrectly predicting scour depths downstream from the weir during flood events, the weir collapsed enabling erosion to progress upstream to the bridge (Fig. 4.5). In order to stop further incision and destruction of the bridge a new larger weir was built (Fig. 4.4) which was designed to promote aggradation and the re-establishment of the original bed elevation in the region of the bridge.

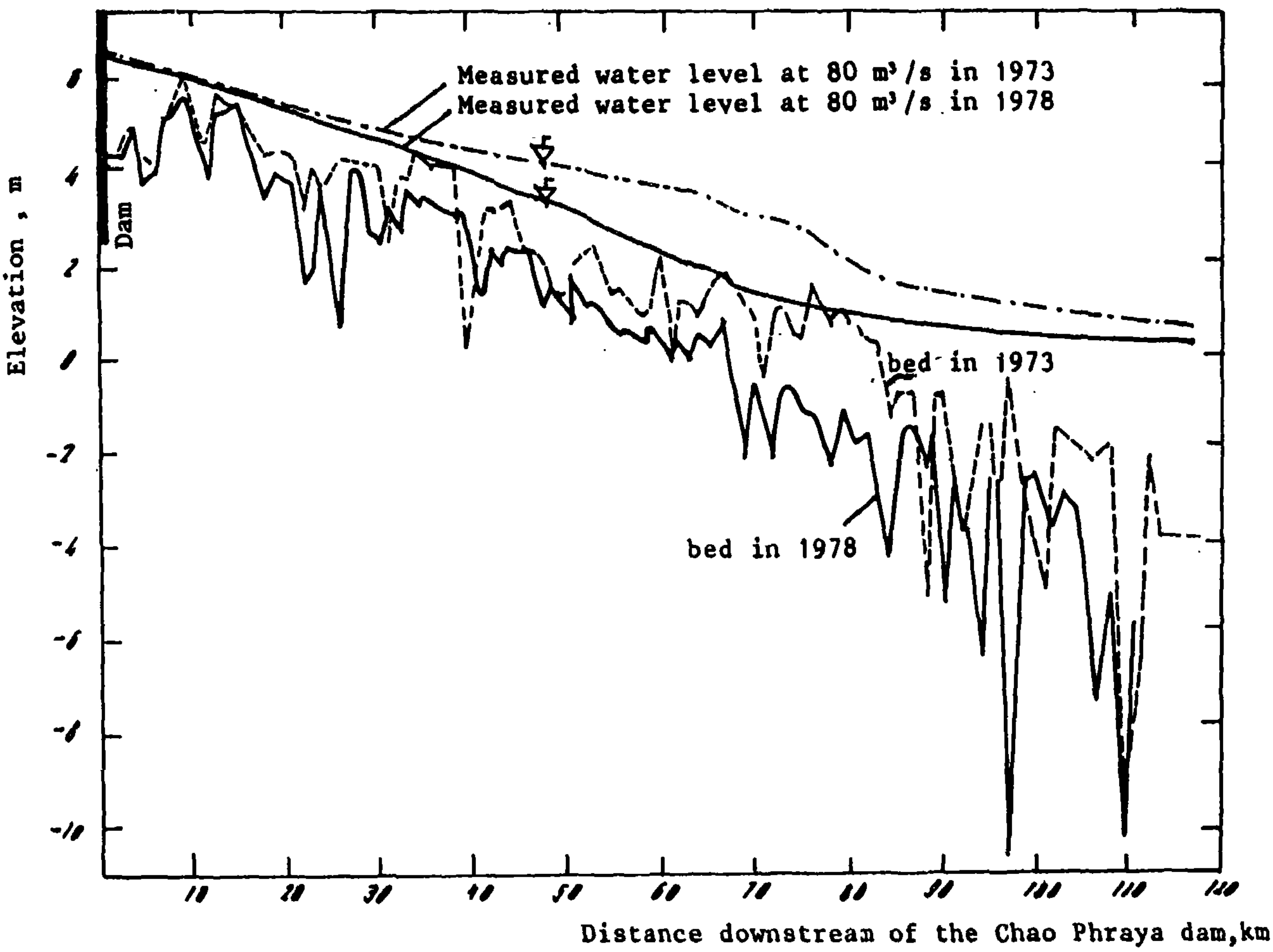


Fig.4.3 Degradation on the Chao Phraya River, Thailand (after Tingsanchal and Overbeek, 1980)

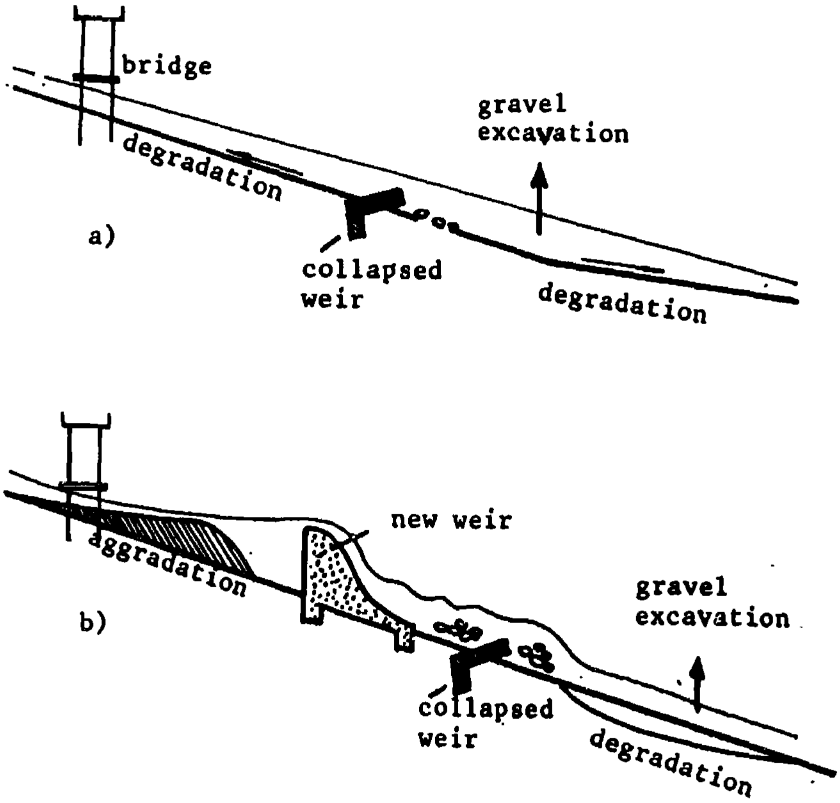


Fig 3.4 Degradational activity in vicinity of bridge pier, river Brenta Italy (after Agostini et. al. 1981)

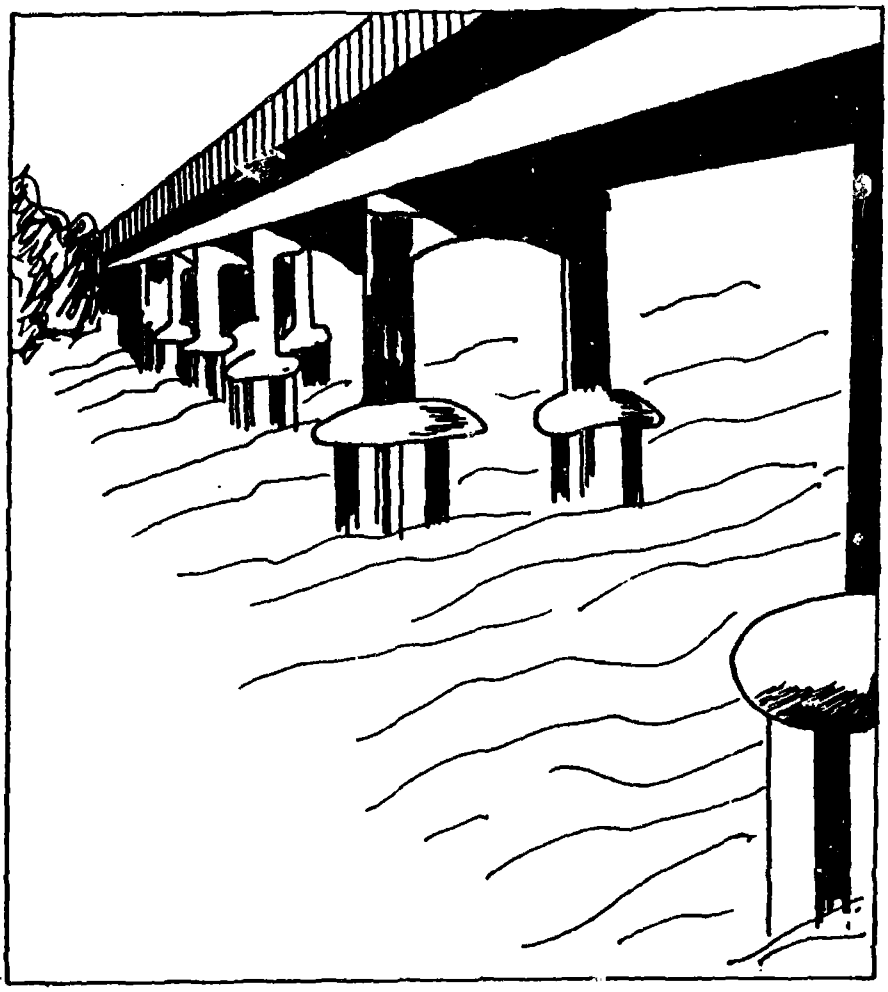


Fig.4.5 Undercutting of bridge piers due to erosion, following failure of a weir, river Brenta, Italy (modified from Agostini et al. 1981).

5. CHANNEL INSTABILITY DUE TO RIVER TRAINING

The training of comparatively small rivers for flood control, navigation and channel stabilization purposes often promotes considerable instability in the river system. For example, modifications to the Willow River in Iowa carried out between 1906 and 1920 converted 45km of meandering river into a straight drainage ditch. Within 4 years the channel had incised 7-8m as illustrated in Figure 5.1.

In some cases incision can affect the stability of bridges, as on the Homochitto River in southwest Mississippi. Modifications carried out between 1938 and 1940 caused rapid degradation for up to 40km upstream which, in the period 1945-74, resulted in the collapse of several highway and railway bridges. Damage totalled approximately $10 million.

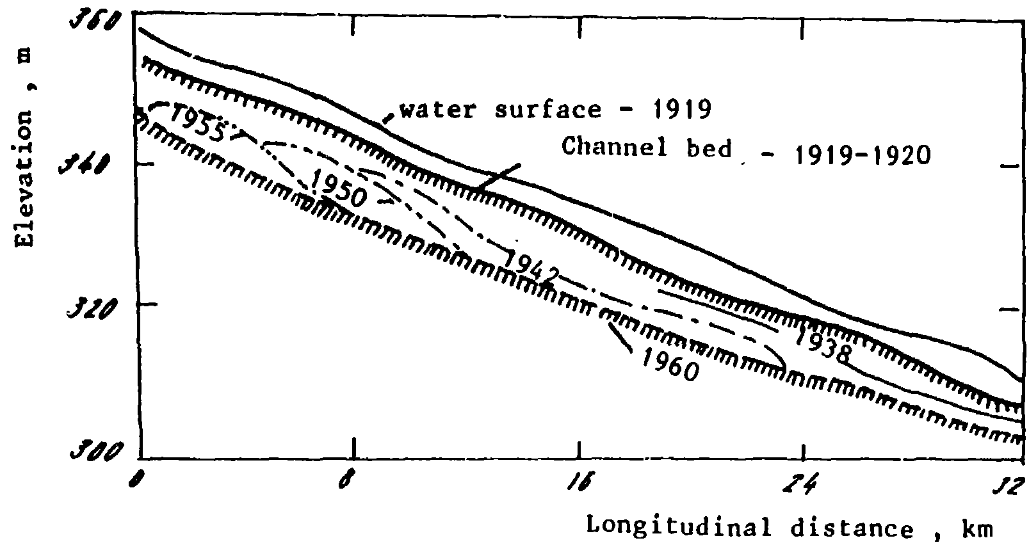


Fig.5.1 Progressive degradation of Willow river drainage ditch, Iowa in the period 1919-1960, due to channelization (after Daniels, 1960).

Erosion on the Mississippi River after regulating a reach 480km long is shown in Figure 5.2. This is based on a comparison of flood profiles before and after channel straightening.

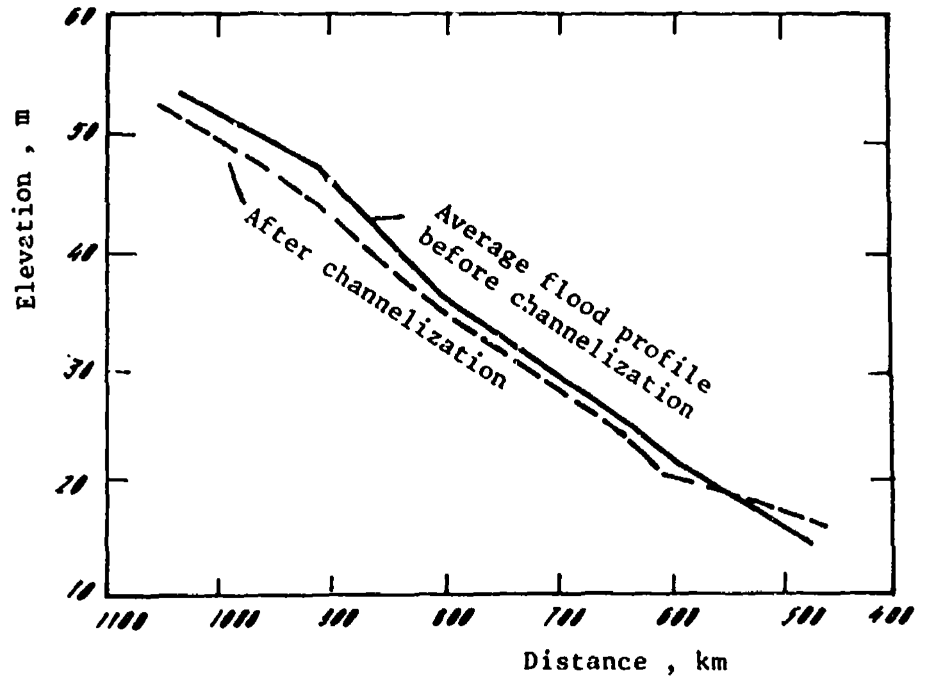


Fig.5.2 Change in average high water profiles as a result of cut-offs, Mississippi river (after Winkley, 1977).

The downstream morphological changes on the river Salzach were studied by Scheurmann, Weiss, Mangeldorf (1980) to identify the effect of training works on channel stability. The Salzach had been unstable prior to river training, being braided between Salzburg (km

66.3) and Laufer (km 47.8) with a width ranging from 1900m to 3800m. In order to establish a fixed boundary between Austria and Bavaria (the river serves as a boundary), create suitable conditions for navigation, prevent floods, etc., training works were carried out in the reach from the mouth of the Saalach (km 59.3) to Laufen (km 47.8) and from Geisenfelden to Tittmoning in the period 1820-1840. This included narrowing of the river bed to 152m by means of bank revetments. Between 1860-1909 the channel width was further reduced to 114m. The training works caused considerable instability. Erosion processes developed in the narrowed reaches due to increased velocities of flow. Some of the eroded material was deposited further downstream. The change in the annual mean water level at Golling, Hallein, Salzburg, Laufen, Tittmoning (Fig. 5.3) indicates where erosion and deposition occurred. No net change in bed elevation was observed at Golling (km 93.6) (Fig. 5.3).

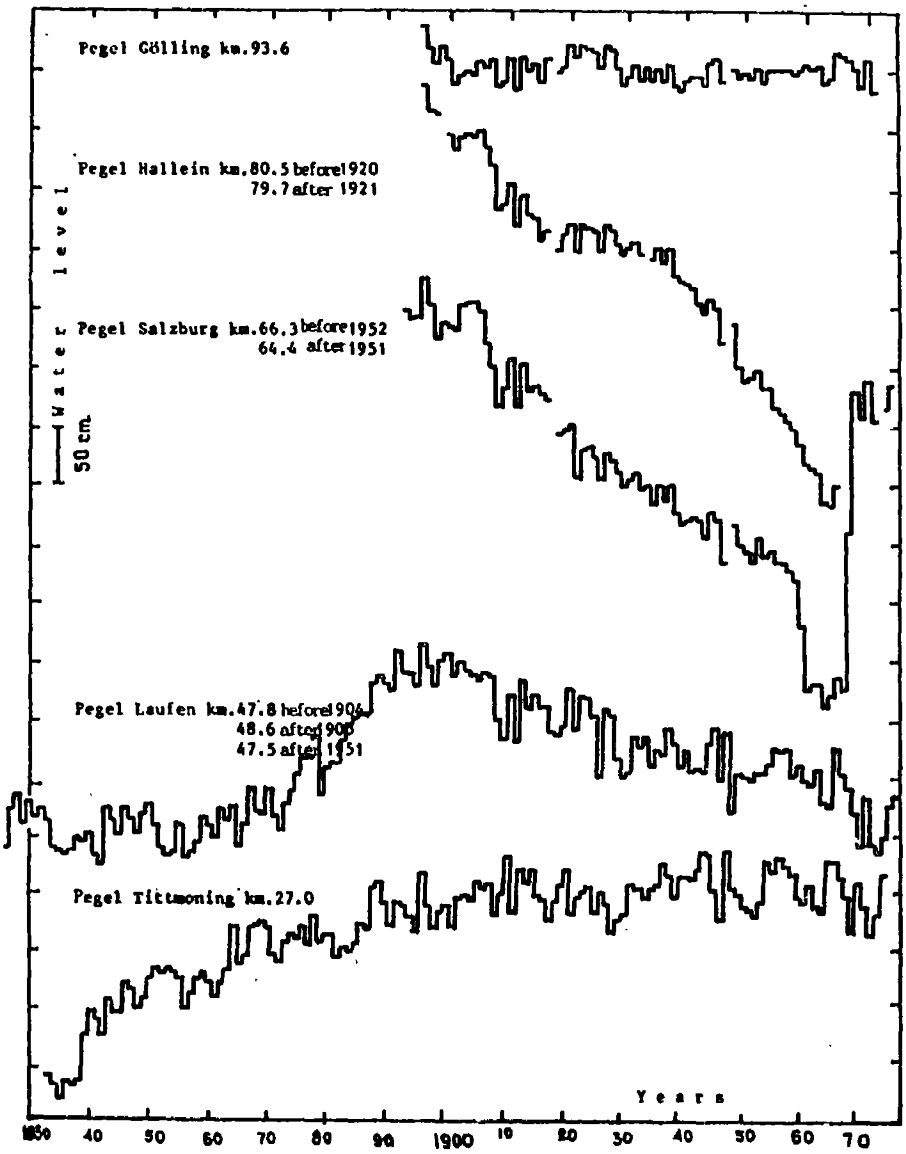


Figure 5.3 Changes in mean annual water levels on the Salzach river at Golling, Hallein, Salburg, Laufen and Tittmoning (after Scheurmann et al., 1980).

Between 1896 and 1965 3.20km of erosion had occurred at Hallein. This was partly due to the training works carried out during the 18th century and also to the reduced sediment loads resulting from the construction of river engineering works upstream on the Salzach River. Degradation was stopped by the construction of a weir at km 804 during 1964-1966. Subsequent aggradation has raised bed levels by up to 4m.

As a result of the training works, progressive incision has occurred at Salzburg (km 66.28, since 1951 at km 64.35). Stage records commenced only in 1893 so erosion rates for before that date have to be estimated. Results indicate that the bed was incised 1.25m between 1848 and 1895 and 2.30m between 1896 and 1944, and an estimated 5.20m for the period 1870-1960.

During a flood event in 1954 incision exposed a weak sand layer. The next flood, in 1959, eroded the bed by 1m at Salzburg, which lead to the destruction of the highway bridge at km 75.2. In 1964 a weir was built at Salzburg at km 64 in order to stabilize the reach between Salzburg and Hallein. Soon after the construction of the weir considerable aggradation occurred and water levels were increased (Fig. 5.3). Prior to 1860 the stage associated with the mean annual water level at Laufen (km 47.8) did not show any systematic change, while between 1860 and 1900 an increase of about 1.5m occurred. This was due to the deposition of material derived from upstream where training works had been carried out. The reduction in channel width from 152 to 114m after 1873 and associated dredging between 1896-1901 did not stop the aggradation. Further dredging was carried out between 1903-1909 and this triggered degradation. Relative stability was observed in the period 1930-1960, while subsequently erosion and a reduction in the annual mean water level has occurred. This is due to the decrease in sediment discharge at Laufen following the construction of Hallein and Salzburg weirs and Urstein dam.

The water level at the Tittmoning (km 27.0) shows a progressive increase between 1838-1920, which is also due to the deposition of material as a result of training works upstream. Major aggradation in the period 1833-1869 is due to training works in the reaches between the mouth of the Salzach and Laufen and between Geisenfelden and Tittmoning. Between 1870-1885 the river at Tittmoning was relatively stable, while between 1885-1905 aggradation occurred. This was due to width reduction upstream from 152-114m, promoting erosion and increasing sediment transport rates.

As a result of long term degradational activity, weak layers were exposed on the bed of the river between km 59 and km 50, and km 37 and km 35. This necessitated the construction of erosion control weirs and the regulation of dredging of coarse material from the river bed.

The creation of a cut-off leads to a shortening of the river length and a local increase in slope of the river (Fig. 5.4). This change can be natural or man-made. The increased slope results in upstream erosion and some of the eroded material being deposited immediately downstream from the cut-off. The rapidity and extent of degradation depends upon the size of the bed material and the hydraulic geometry of the channel. In time, headward erosion will increase slope in this upstream section.

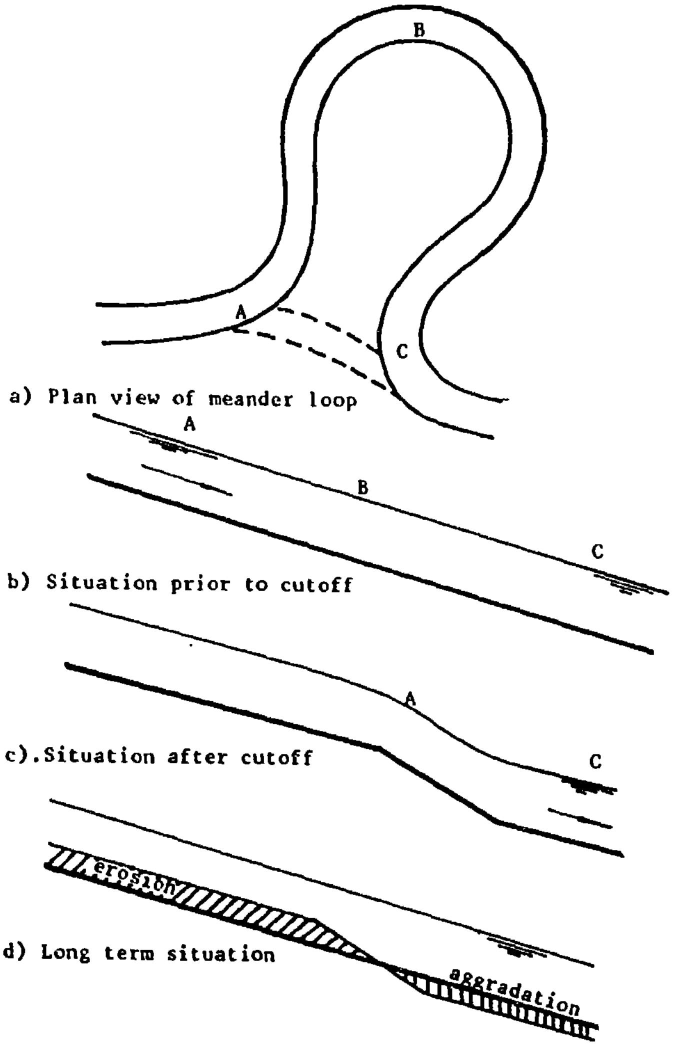


Fig.5.4 Headward erosion initiated by a cut-off (after Galay, 1983).

Field studies of the development of an artificial cut-off on the Yang-Tse River have been carried out by Pau Ching-shen, Shin Shoa-Chaun and Tuan Wen Chung (1978). Two meanders were cut-off in the middle part of the Yang-Tse River in order to shorten the navigation route and increase the flood capacity of the channel (Fig. 5.5). The mean annual discharge of this reach of river is $303,000.10^3 m^3$, the sediment run-off is $290.10^6 t$ and the average suspended sediment concentration is 0.96 kg/m^3/yr.

The upstream, first cut-off was 4.3km long, compared with 36.7km for the original meander bend and was constructed with a bed width of 30m and a depth of 6m. The latter corresponding to the depth of the alluvial deposits in this reach. The second bend, 32.7km long, was reduced to 3.5km by the cut-off channel. This channel also had a bed width of 30m, and a depth of 6 m, while the flood plan deposits varied in thickness from 30m at the upstream end to 4-6m at the downstream section. The median bed material size in both reaches was coarse sand (D_{50} = 0.17mm). The cross sectional area of the first cut-off channel was 1/30 of the original channel area; the area of the second varied between 1/17 and 1/25 of the original sectional area.

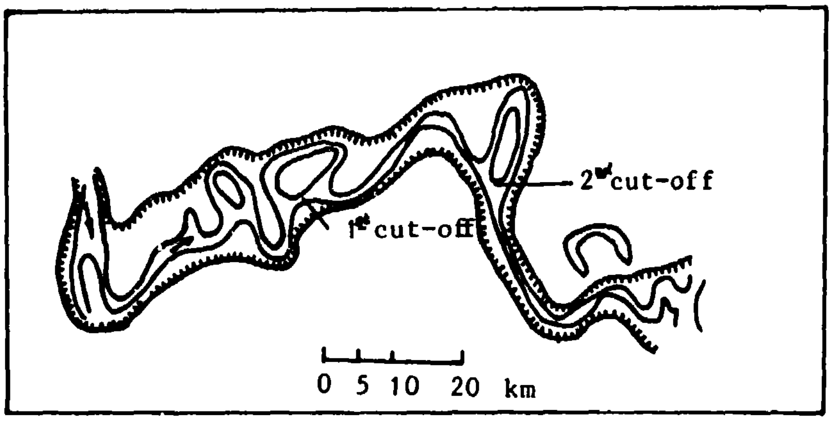


Fig.5.5 Location of artificial cut-offs in the middle Yang-Tse river (after Pau Ching-shen et al., 1978)

3 stages can be recognised in the subsequent development of these cut-offs: first degradation, second meander transformation and third ox-bow lake formation. At the beginning of the first stage a small part of the streamflow passes through the steep cut-off section. Eventually a larger part of the streamflow passes through it, while the water surface slope declines as a result of headward erosion. At the end of the first stage, when the slope is roughly twice its original value, approximately 50% of the total streamflow passed through the first cut-off (Fig. 5.6). During the second stage rapid siltation occurs in the meander bend, and up to 90% of the total flow is transmitted by the cut off channel. At the end of the third stage, the meander bend is isolated from the cut-off channel and an oxbow lake is formed.

Changes in the flow depth, width (both absolute and relative values) and roughness in the cut-off are of considerable interest. Flow depth rapidly stablized, while width progressively increased (Fig. 5.7). The width of both cut off channels increased rapidly, with the first being considerably wider than the second. Seasonal trends in width are noticable at both sites reflecting the influence of flood flows.

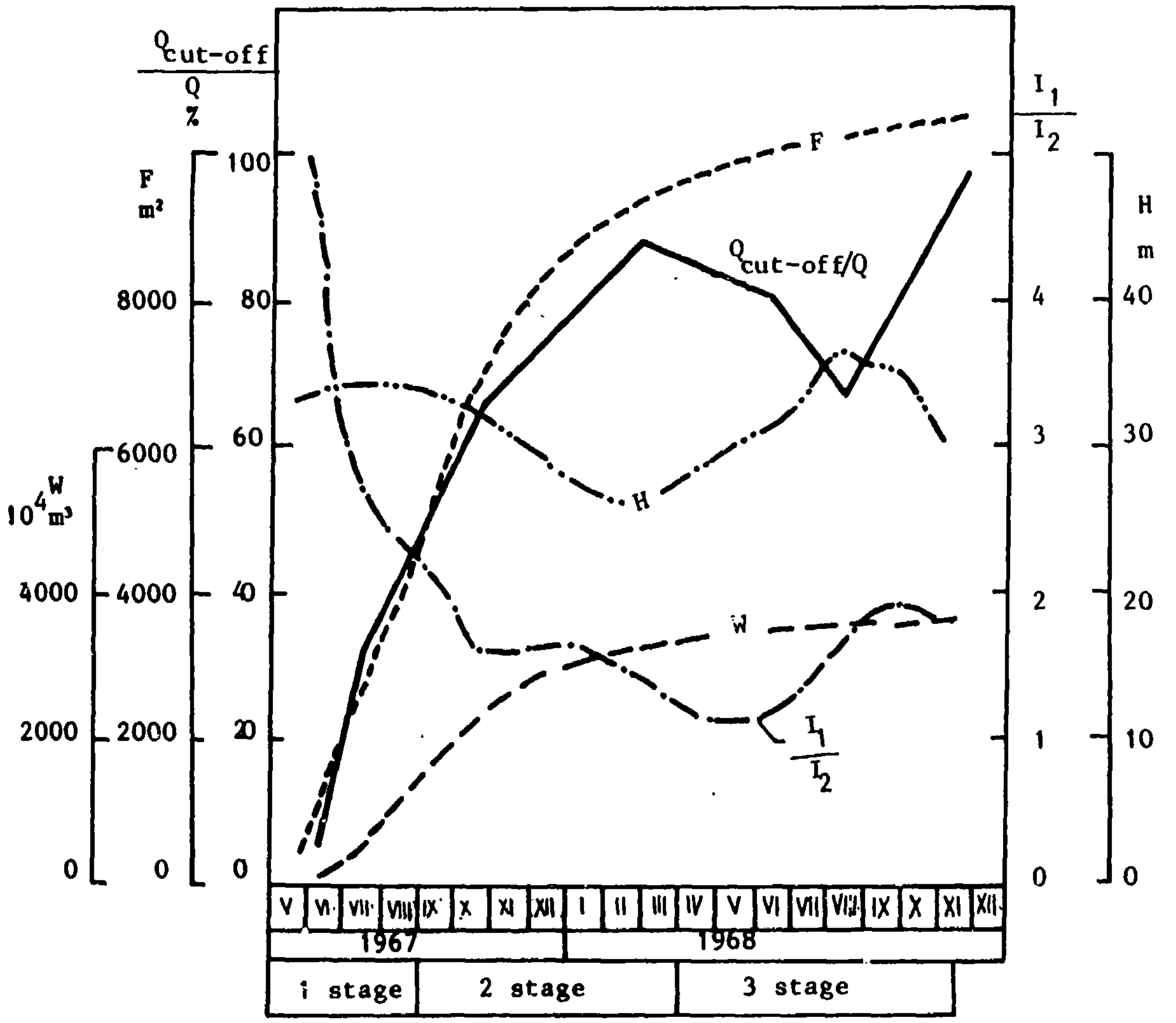


Fig.5.6 Changes in hydraulic geometry during the three stages in the development of first cut-off, Yang-Tse river (after Pau Ching-shen et al. 1978) (total volume eroded W, cross sectional area F, total discharge Q, discharge in the cut-off channel Q cut-off, bed slope in cut-off I_1, bed slope downstream from the cut-off I_2, water level at the entrance of the cut off H)

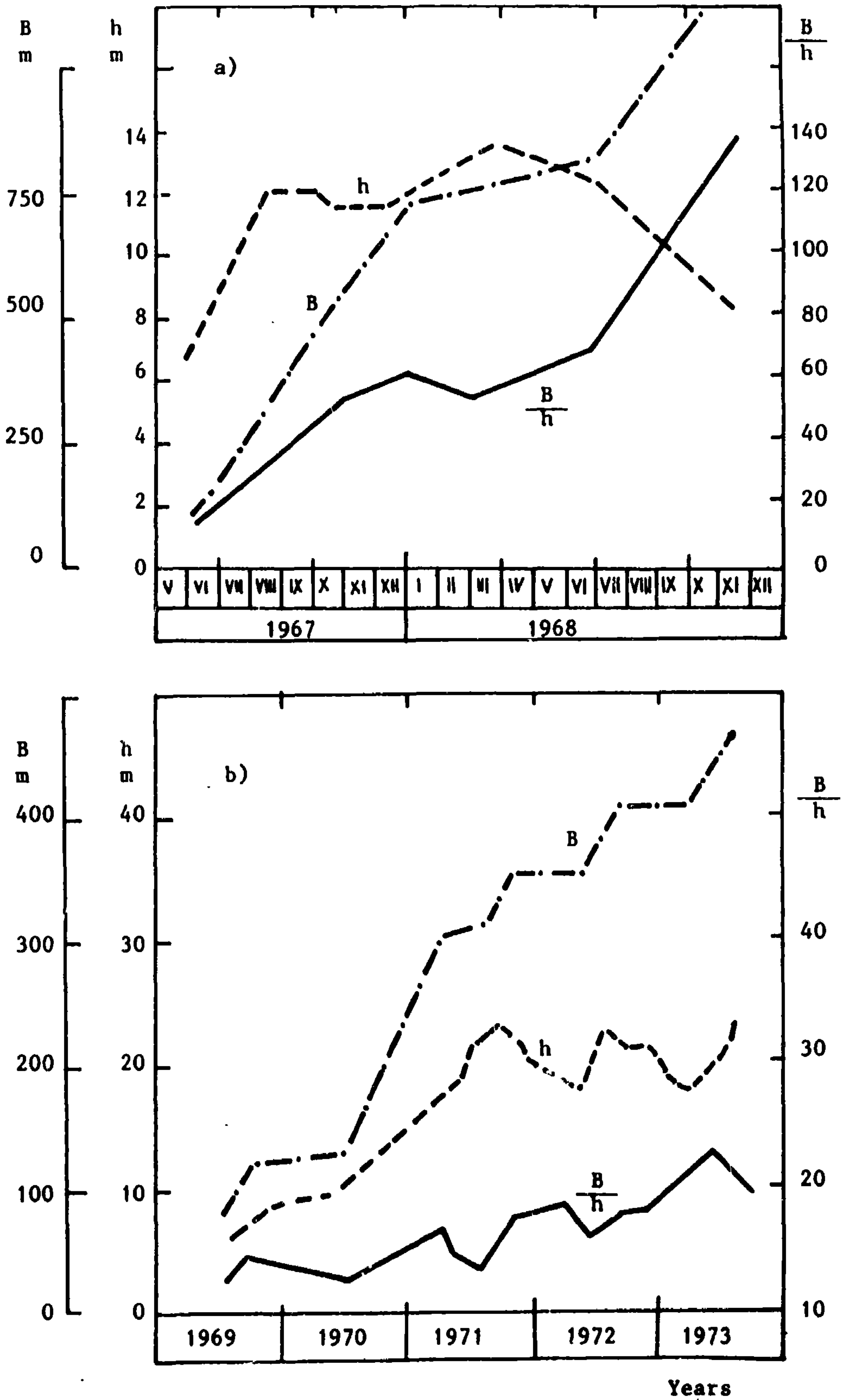


Fig.5.7 Variations in the width and depth of the first (a) and second (b) cut-offs (after Pau Ching-shen et al., 1978) B - width, h - depth)

There are also three stages in the development of the meander bend post channel straightening. Decreases are observed in cross-sectional area, discharge and water surface slope during the first stage, and this encourages the deposition of sediment in the meander loop. Nevertheless, during this period natural patterns of seasonal erosion and deposition are preserved in the bend. At the end of this stage the discharge in the meander loop is 50% of its original value.

The second stage is characterized mainly by sedimentation and seasonal patterns of erosion and deposition cease. The discharge in the meander loop is less then 10% of the flow in the main river.

An ox-bow lake is formed in the third stage. The entrance of the cut-off silts up before the exit section. When deposition in the entrance has virtually ceased, the exit rapidly silts up due to the action of reverse currents. Thus, as long as the entrance of the old meander loop is not completely filled, sediment discharge in the bend gradually decreases downstream. Immediately after the entrance section is blocked to a level, for example, corresponding to the average flood level, the zone of maximum deposition shifts to the exit section leading to the formation of an isolated ox-bow lake. The rate of sedimentation in the meander loop is shown in Figure 5.8 and Figure 5.9.

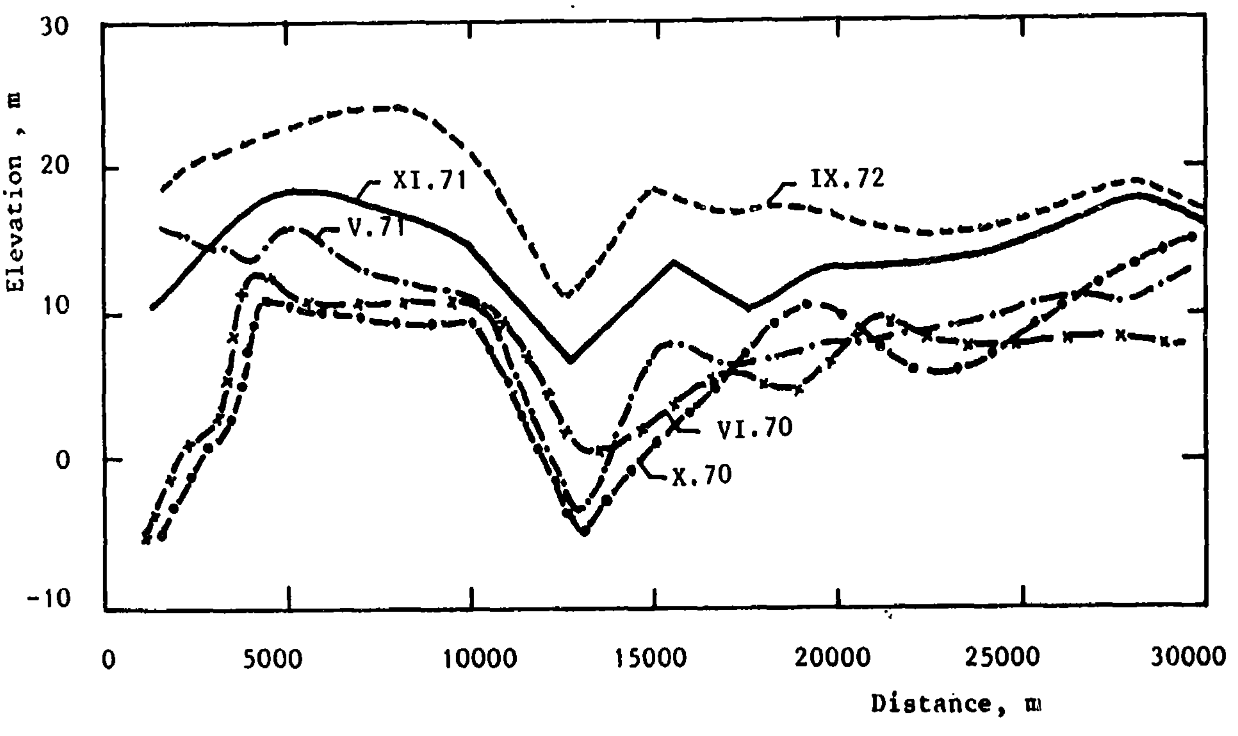
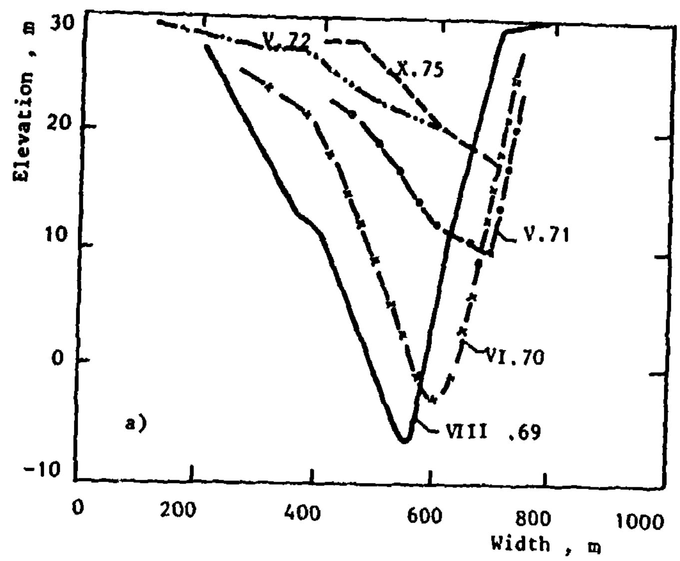


Fig.5.8 Downstream aggradation in second meander bend (after Pau Ching-shen, 1978)



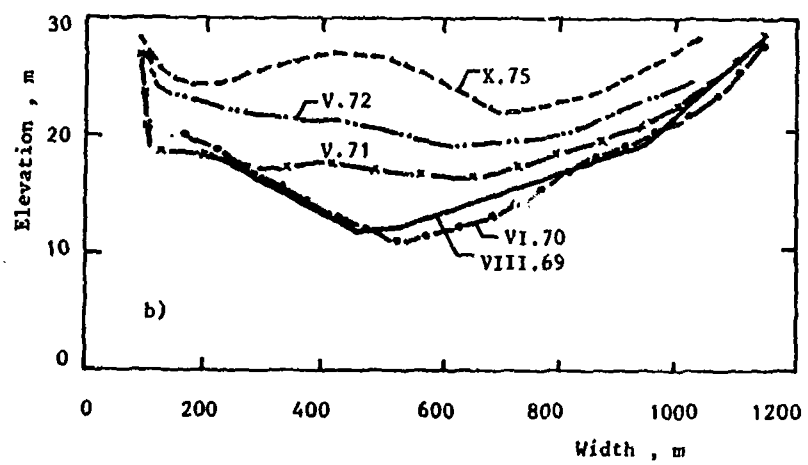


Fig.5.9 Aggradation at a) pool and b) riffle section in second meander bend (after Pau Ching-shen, 1978)

The cut-offs also affected water levels upstream, these being reduced by 0.32, 0.42 and 0.59m at 136 km, 69 km and 42km upstream from the first cut-off.

6. COMBINED EFFECT OF RESERVOIRS, DREDGING, RIVER TRAINING AND FLOODS ON CHANNEL STABILITY.

Multipurpose water resource development schemes involving river engineering works relating to water supply, power generation, irrigation, navigation and flood alleviation can all adversely affect the stability of natural channels. While it is possible to identify the effect of each type of development on the river within any one basin it is usually necessary to consider their joint effect on the river system. Galay (1983) suggested a scheme which enables the erosional effect of various combinations of river engineering works to be considered (Fig 6.1).

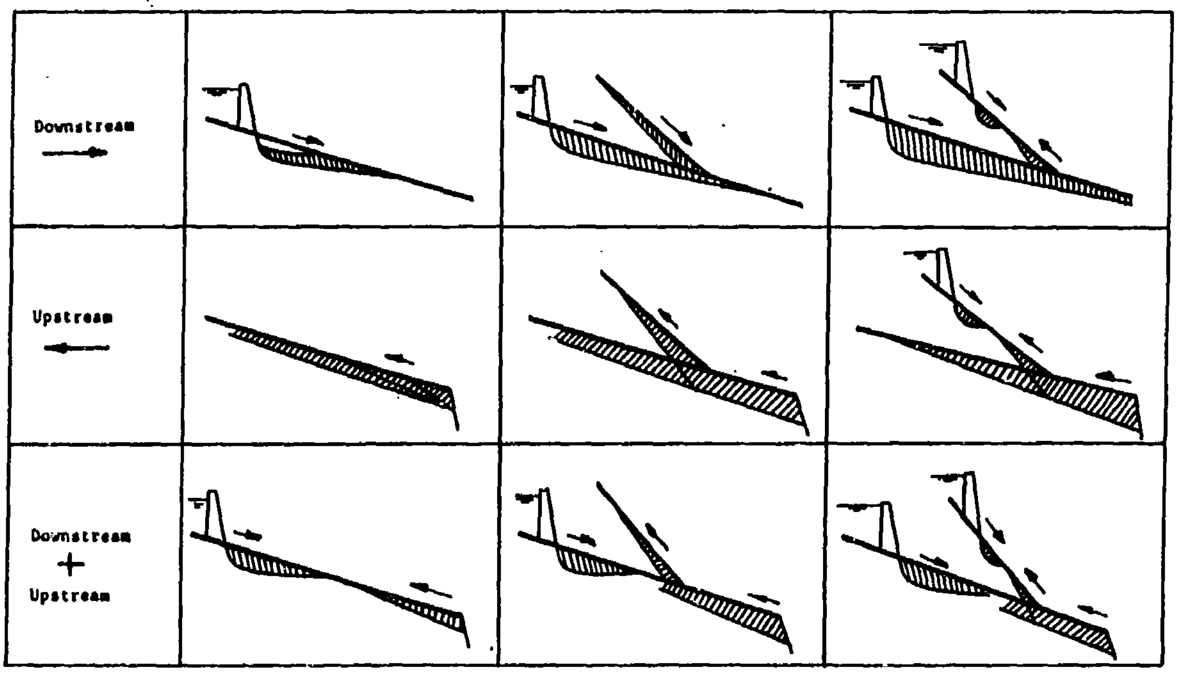


Fig.6.1 Effect of various combinations of river engineering works on erosion.

The river Maritza (Bulgaria) is a typical example where channel instability has resulted from the combined effect of dams, river training, dredging and weir construction. In the Maritza basin 8 reservoirs have been built (Fig. 6.2). Dredging has also been carried out at seven sites, the amount removed annually being 20 times the natural annual sediment yield (pre dam period). Training works have also been implemented, often involving weir construction (Fig. 6.2).

As a result of these changes erosion has occurred producing a systematic change in stage-discharge relations (Fig. 6.3). Temporal changes in water level for the mean annual discharge are also indicative of channel instability. Information for four sites on the Maritza is given in Fig 6.4.

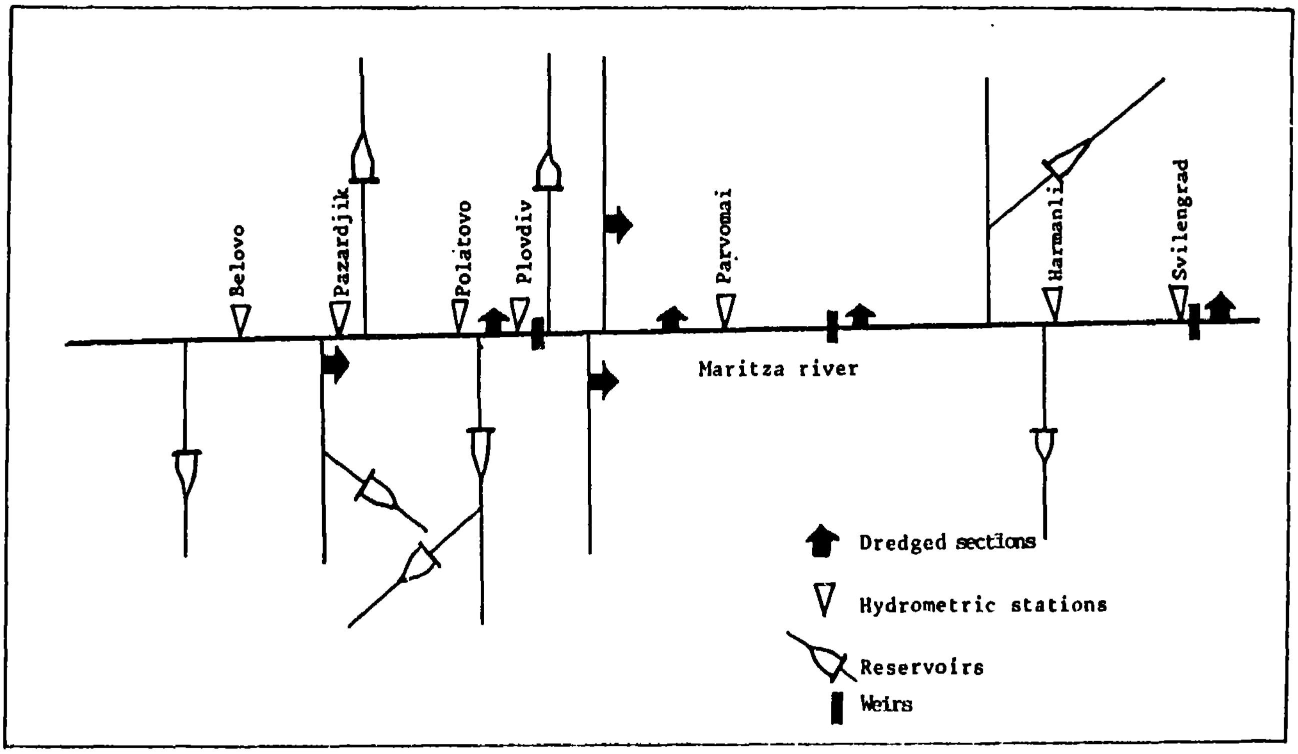


Fig.6.2 Location of reservoirs, dredged sections, weirs and hydrometric stations on the river Maritza (Bulgaria)

The cessation of erosion at Plovdiv is a result of the construction of a weir upstream from the dredged reach which reduced the water surface slope and transport capacity of the river. Rapid degradation at Svilengrad, which threatened to destroy the existing main line railway and road bridges was halted by the construction of a weir between the dredged section and the bridges. The weir increased water levels upstream and promoted aggradation which protected the existing structures.

There are numerous examples where the combination of several factors have affected the stability of Alpine rivers in the Federal Republic of Germany. Initially several braided reaches were canalised. This being followed by some dredging for aggregates and the construction of a number of low head hydro-electric power stations. Each of these changes had a significant local effect on sediment transport rates which temporarily destabilized some sections of river. In the process of attaining a new equilibrium, surface armouring can occur. However during large floods this may be breached, causing further instability until a new armour layer is established. When a multiplicity of factors affecting channel stability are altered by man's activities, great care has to be exercised when predicting equilibrium conditions, especially when bed armouring is involved.

The combined effects of canalization, dredging and dam construction is well illustrated by changes that have occurred on the Lech River (Federal Republic of Germany). Figure 6.5 illustrates the reduction in water levels on the Lech at Schongau, Landsberg, Schwabstadt, Lechhausen and Meitingen as a result of canalization, dredging and

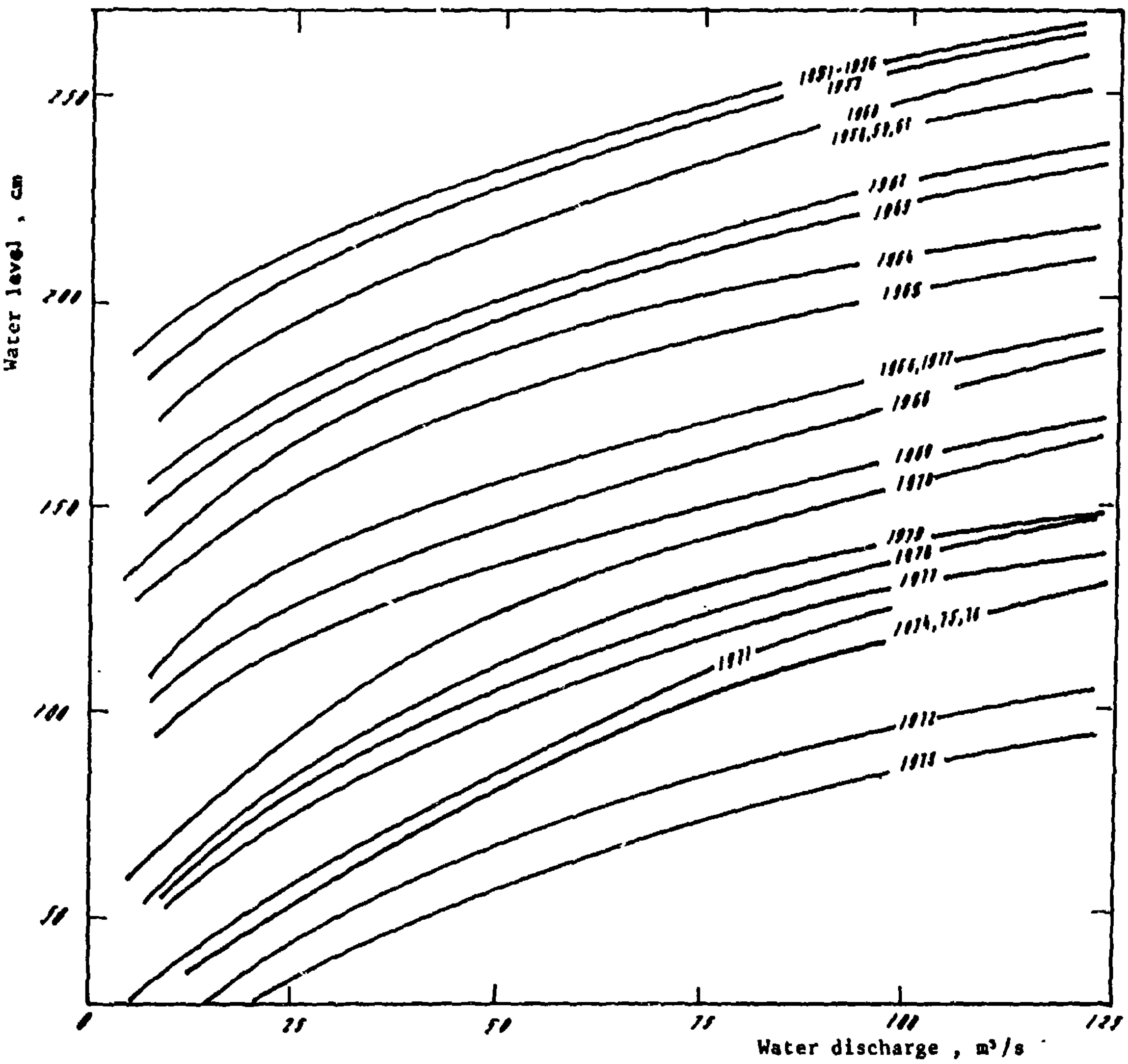


Fig.6.3 Stage/discharge curves at Pazardjik, river Maritza (Bulgaria) for the period 1951-1979

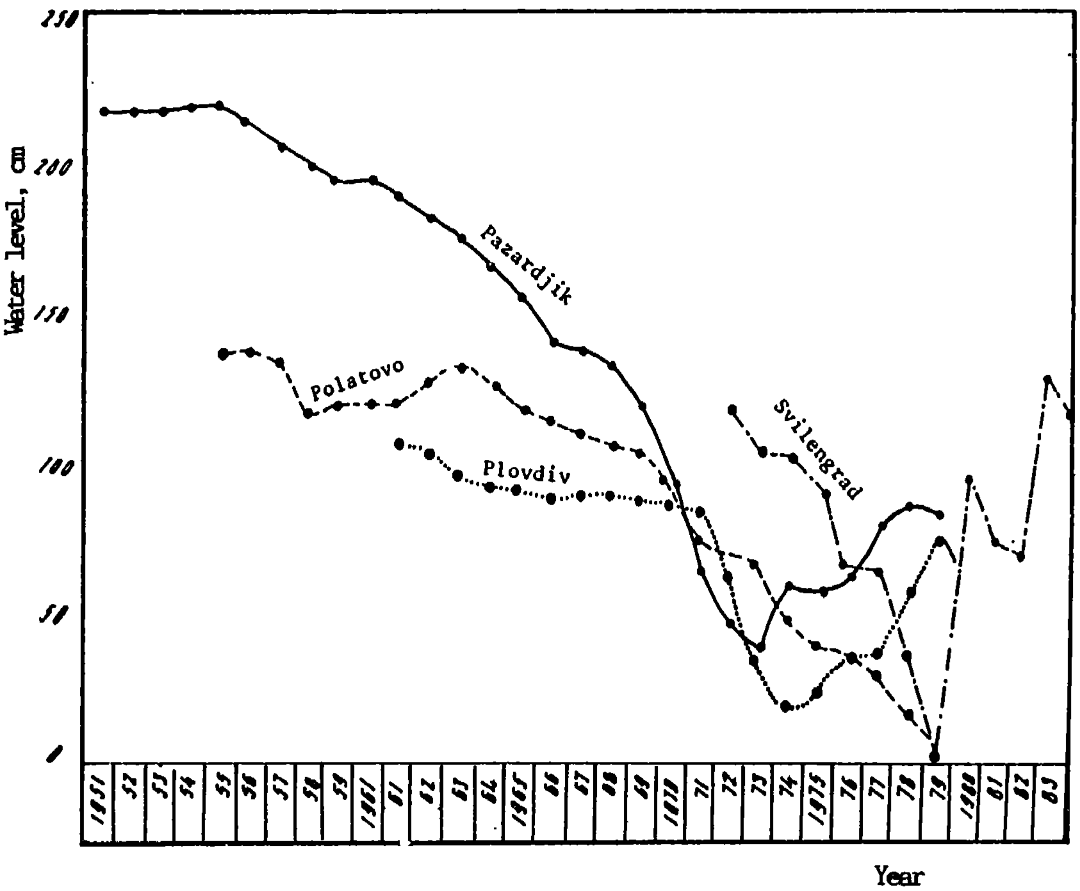


Fig.6.4 Change in water levels corresponding to the mean annual discharge at Pazardjik, Polatovo, Plovdiv and Svilengrad,

flooding. After canalization works commenced degradation began which lead to the periodic formation and destruction of an armour layer. After the floods in 1910, characterized by a peak discharge of 1100m^3/s, progressive degradation occurred which by 1930, had armoured the bed of river. No further erosion took place until 1970. Changes in the cross section of the Lech at Gersthoffen indicates that the bed was scoured by an average 6-7m (Fig. 6.6 and 6.7) between 1880 and 1970.

Similarly on the river Rhein a variety of river engineering works, including canalization and the construction of depth control structures for navigation in the upper reaches, have caused degradation further downstream. Water levels associated with the mean annual discharge have progressively declined since 1820, and

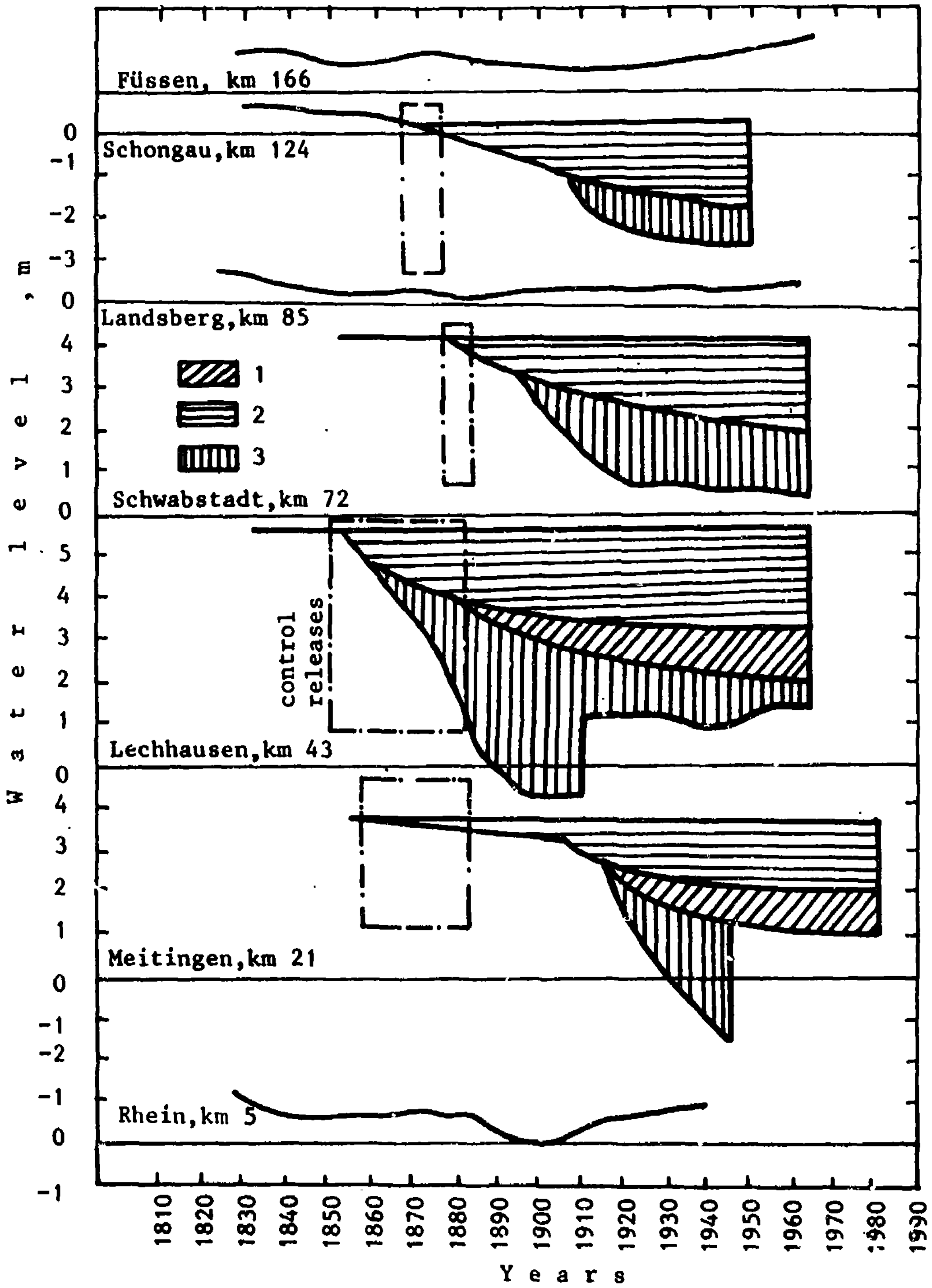


Fig.6.5 Reduction in water levels at Füssen, Schongau, Landsberg, Schwabstadt, Lechhausen and Meitingen on the river Lech (FRG), due to 1) dredging, 2) river training, 3) floods (after Bauer, 1979)

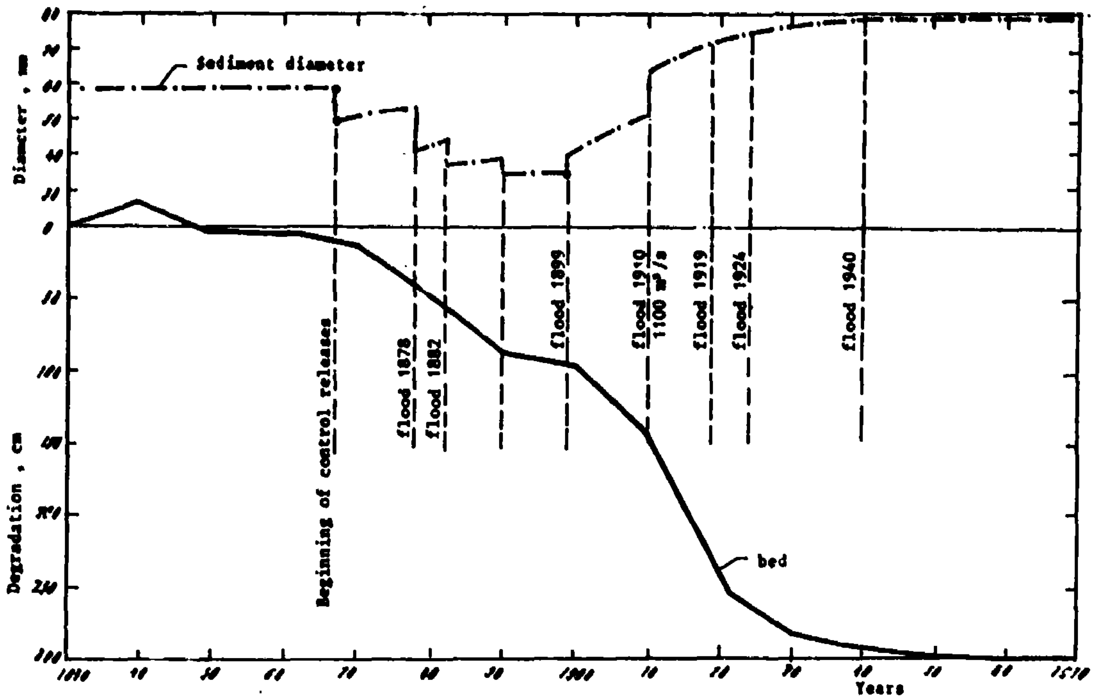


Fig.6.6 Bed material diameter and depth of degradation at Schongau on the river Lech (FRG), due to river training and floods, (after Bauer, 1979)

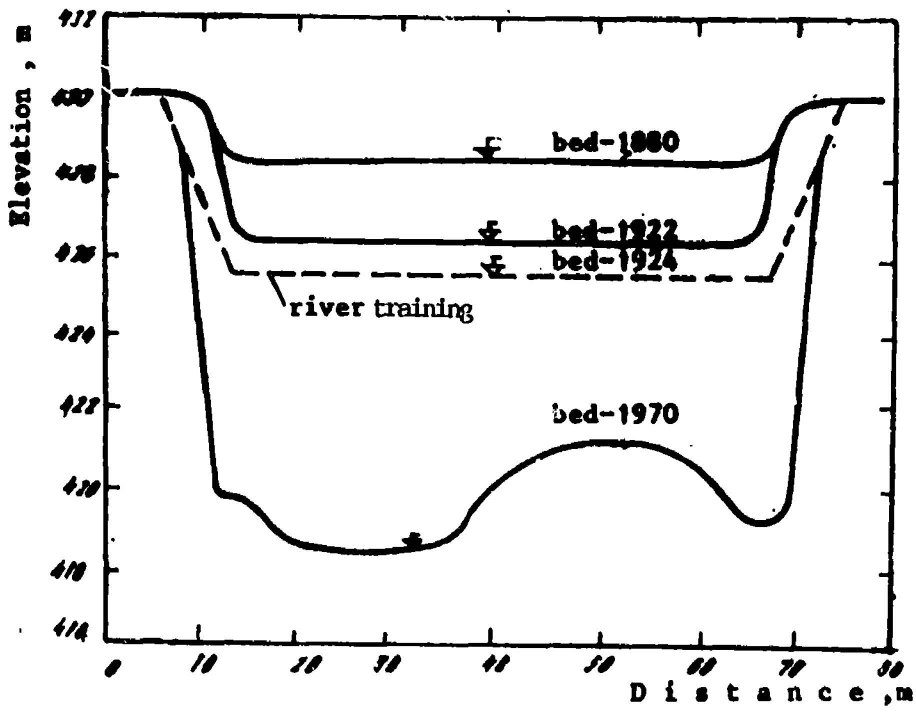


Fig.6.7 Progressive degradation at Gersthoffen, river Lech (FRG) between 1850-1970 (after Bauer, 1979)

particularly since 1900, and are now being reduced by as much as 3.1cm/year (table 6.1).

TABLE 6.1 Reduction in water level associated with mean annual discharge on the Lower Rhein (after Eschweiler 1952)

Point	km	Period 1820-1900		Period 1900-1950	
		total cm	per year cm	total cm	per year cm
Köln	688	54	0,4	36	0,7
Düsseldorf	744	112	0,9	97	1,9
Ruhrort	781	213	1,6	148	3,0
Wesel	814	248	1,9	158	3,1
Rees	837	181	1,4	114	2,3
Emmerich	853	133	1,0	69	1,4

Determining the influence of each factor on erosional and depositional activity and quantifying the parameters controlling these processes is exceedingly difficult. Any attempt at a solution is dependent on the availability of long term systematic observations of river flows, associated transport rates, calibre and nature of the bed and bank material and the morphology of the channel not only at the affected site, but also upstream and downstream from it.

7. PREDICTING CHANGES IN CHANNEL PATTERN

Erosional and depositional activity in alluvial rivers, produces seven different types of channel pattern, according to a conceptual morphological model of channel evolution (Fig. 7.1). The direction of the arrow shows a suggested increase in the transport capacity of the river.

Channels with transverse bars are typical of intermediate and small alluvial rivers composed of medium and coarse sands, as well as mountain and piedmont reaches and some branches of large alluvial rivers. It is characterized by separate transverse bars; their length being 6-8 times the width of the channel and their height is 0.15-0.30 of the flow depth in the pool during flows with a frequency of less than 10%. Channel instability results from the migration of the bars downstream which produces periodic aggradation and degradation at a particular site. Their presence can be determined by echo sounding, visual air reconnaissance and from aerial photographs of the channel taken during low water periods. This type of river is characterized by the absence of a flood plain.

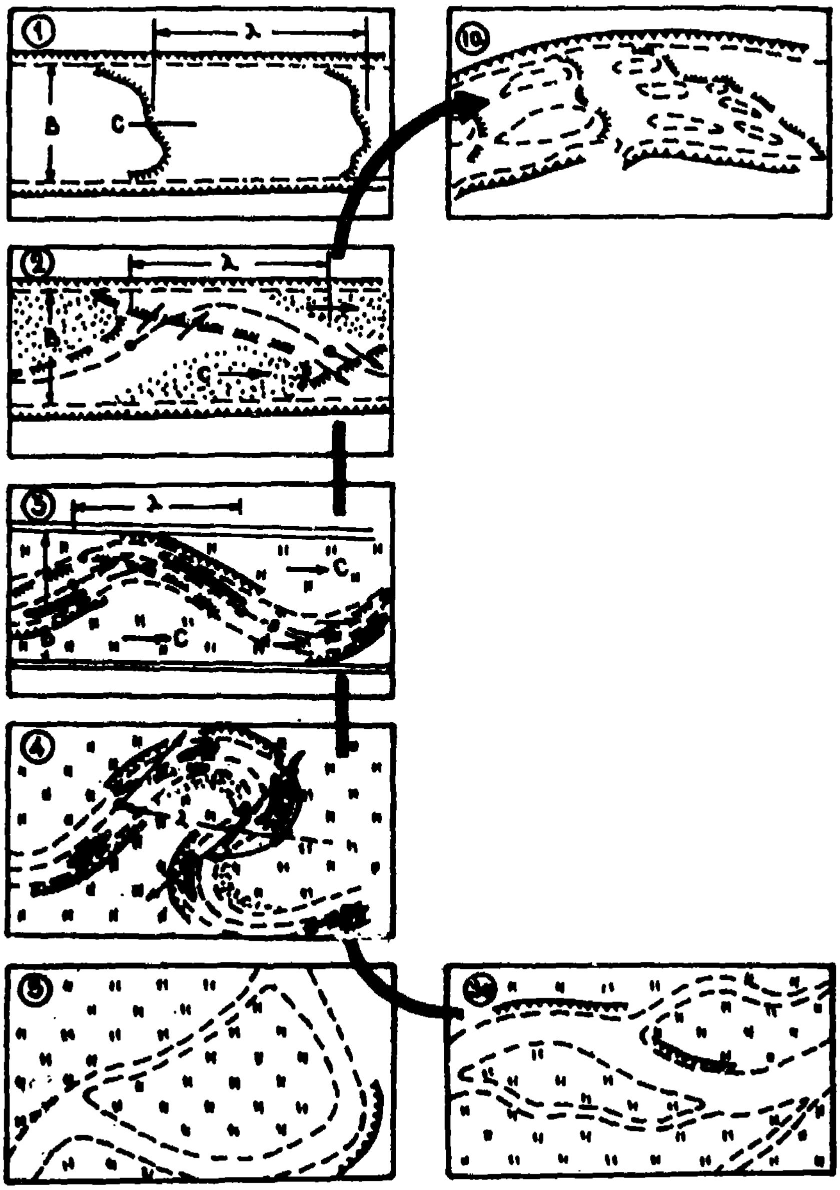


Fig.7.1 Types of channel pattern

1 - channel with transverse bars (λ - spacing of transverse bars);
2 - lateral bar (λ - spacing of lateral bars);
3 - restricted meandering (λ - distance between inflexion points);
4 - free meandering (λ - distance between inflexion points);
5 - partial or noncomplete meander;
1a - channel with medial bars; 5a- braided;
(after Kondratiev et al., 1982);

Channels with alternate lateral bars are widespread in alluvial and piedmont rivers irrespective of bed sediment size. In general its occurrence is restricted to straight and slightly meandering channels although it can be found in some sections of rivers with medial bars and in meandering channels. It is characterized by large shoals, located at alternate sides of the channel, which may be partially exposed during low water. During floods the shoals are submerged under water and the channel looks straight while during periods of low flow the channel is sinuous. Pools in the channel are located adjacent to the convex face of the bars, or between bars in the meander bend. The bar form migrates downstream during floods, maintaining a height of 0.3 H_p, where H_p is the bankfull flow depth and a length of (4-8)B, where B is the mean width of the channel. Characteristically rivers of this type do not have a floodplain and bank erosion is relatively minor and random in location.

Restricted meandering is often observed on rivers with flood plains. Although meander arc angles, measured between inflexion points, can be as large as 120°, free meander development is limited by the non-erodible valley side slopes. The thalweg is sinusoidal and the meander pattern is not drowned out during flood flows. Morphologically this form is similar to that of the lateral bar type with the bars developing into small isolated sections of flood plain (Fig. 7.1). Bank erosion downstream from the apex of the meander bend and associated deposition on the point bar further downstream results in the downvalley migration of the meander pattern without significant change in its plan geometry. Scour and fill activity is similar to that in channels exhibiting alternate lateral bars. During low flows the shoals are subject to erosion, while during floods they aggrade. The pools scour during flood events with aggradation during the period of flood recession. Bar deposits have two components: a lower layer of relatively coarse material, comparable in size to the bed material and resulting from the deposition of bed load, and an upper layer of fine sand, silt and clay which is derived from the suspended sediment load. Levees may occur adjacent to the channel where reduced velocities at the interface between channel and overbank flow encourages preferential deposition.

Free meandering is very common on rivers with wide flood plains. It is characterized by a single channel, like restricted meanders, but erosional and depositional processes are more complicated as the river can adjust its plan geometry.

Initially, when meander arc angles are less than 90°, the meanders migrate downstream and arc angles tend to increase. This reduces the rate of downstream movement with the consequence that the shape of the meander bend changes (bends are stretched). At arc angles of about 140° a meander cut-off occurs. The local increase in channel slope in this reach encourages further instability as the river responds to the new conditions (Chapt. 5). Seasonal scour and fill activity similar to that for restricted meanders, can occur although it may not have a permanent effect on channel evolution. Bedrock outcrops, or deposits which are resistant to erosion, can modify the evolution of the channel pattern.

The flood plain is formed by the deposition of bed and suspended load on the point bars. Individual flood events may produce a series of ridges on the bar surface as the meanders migrate downstream which can be reflected in the topography of the flood plain. Ox bow lakes, at various stages of infill, are also characteristic of freely meandering rivers.

Partial or noncomplete meandering, a variant of free meandering, occurs when a chute channel is formed across the flood plain surface and the river is straightened. It occurs at sections where the flood plain deposits are easily eroded and develops during major floods. During the initial stage of free meander development a chute channel is cut in the flood plain across the meander bend which later develops into the main channel. The straightened section is often subject to the rapid development of transverse, medial and lateral bars. Eventually the chute channel transmits most of the flow and an ox bow lake is created and the cycle continues. This evolutionary process occurs rapidly on small rivers, but more slowly on large ones.

Braided channels evolve from partial meandering ones when chute channels form on several adjacent bends. All types of bar forming processes can operate in each individual channel. Island formed by the braiding process have the same surface elevation as the adjacent flood plain. As a result of bar development the river is continually changing its patterns, and temporary channels can form during flood events.

Channels with medial bars are common in flood plain, piedmont and mountain rivers which transport high sediment loads. Rapid aggradation produces medial, lateral and transverse bars which, during low flow periods give the channel a braided appearance. In river reaches composed of fine sediments the medial bars are unstable at low flows in contrast to rivers flowing in coarse and medium sized bed material. Colonisation by vegetation can encourage the bar to develop into an island. The deposition of fine sediment on medial bars also reduces their mobility.

Consideration of channel response to erosional and depositional processes, based on the conceptual model outlined and the above results of empirical investigations provide a basis for assessing the response of the river to any imposed changes.

A useful empirical relation has been established for rivers in the USSR between channel morphology, valley slope and flood discharge (fig.7.2) (Romashin 1968). This shows that for an increase in discharge or valley slope (the latter being related to sediment discharge) channel form changes from free to partial meandering and eventually to a braided state. The graph is valid for rivers which have no lateral constraint.

For $Q\,S_v \geq 350$ free meandering is observed, for $350 \leq Q\,S_v \geq 1400$ partial meandering occours and for $Q\,S_v \geq 1400$ the channel either has medial bars, for large valley slopes, or is braided, for high flood discharges. This graph may be used for predictive purposes to assess the effect on channel pattern of changes in discharge due to river engineering works. If flood runoff is reduced by regulation, there may be a transition from braided through non-complete meandering to free meanders, or from straight channels with medial bars through partial/non-complete meandering to free meanders. This has been proved in practice.

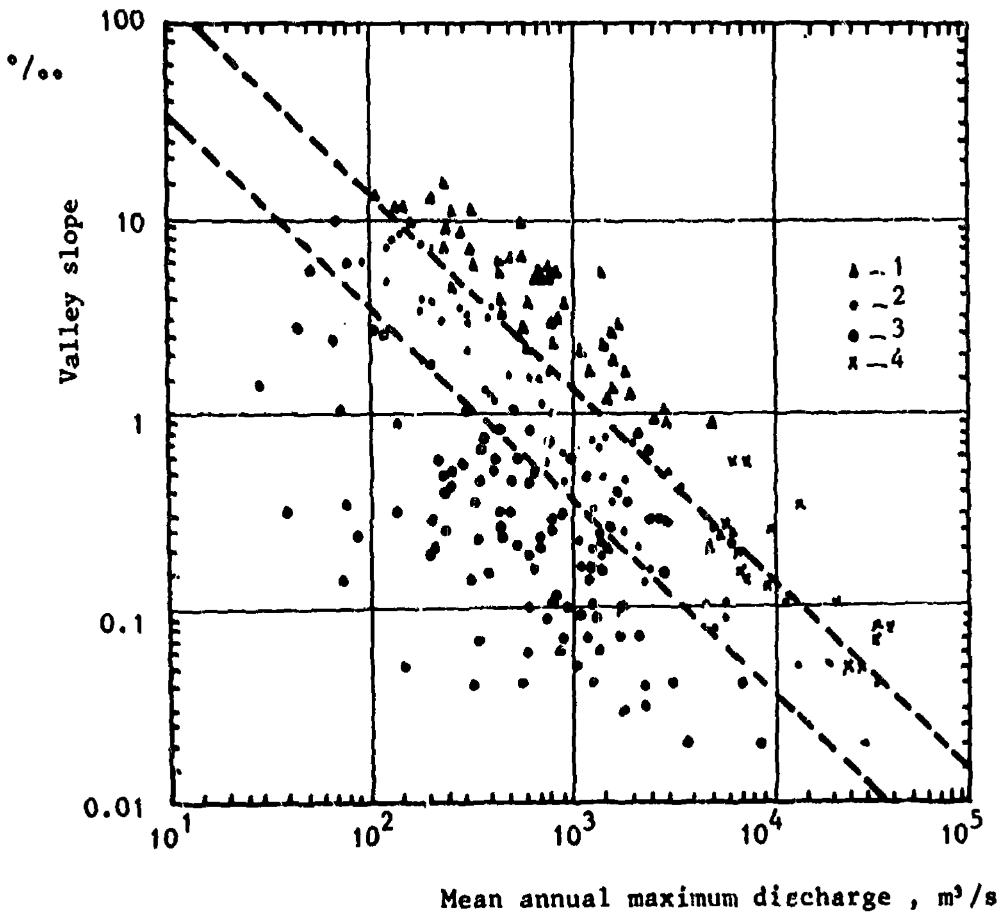


Fig.7.2 Dependence of channel pattern on valley slope and mean annual maximum discharge
1 - channels with medial bars; 2 - partial meandering; 3 - free meandering; 4 - braided channels; (Romashin, 1968)

Where engineering works limit the width of the channel or its flood plain, an approach developed by Snishchenko (1979) can be adopted. If the active flood plain width is given by B_o, the amplitude of the channel pattern B_a, channel width at low flow B, valley slope I_o and channel slope I, then the ratios B_o/B_a, B_o/B I_o/I characterise the channel pattern. By definition for a given value of channel width, B, straight channels have narrower active flood plains than braided and meandering ones (fig. 7.3). The critical ratios of B_o/B at which channel pattern changes are listed in table 7.1. Similarly, critical values can be found for the ratios of B_a/B.

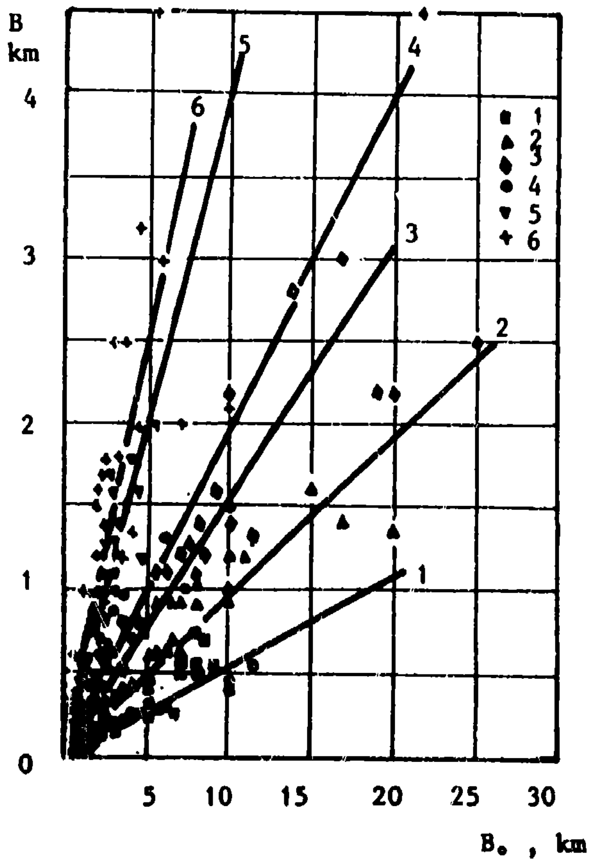


Fig.7.3 Effect of channel width (B) and valley width (B_o) on channel pattern.

1 - free meandering; 2 - noncomplete/partial meandering; 3 - braided; 4 - restricted meadering; 5 - channel with lateral bars; 6 - channel with medial bars. (Kondratiev et al., 1982).

TABLE 7.1 Ratios of channel to floodplain widths for different types of channel pattern

Type of channel pattern	Mean / Standard	
	B_o/B	$B_{c.b.}/B$
Free meandering	18.30/4.57	8.86/2.53
Partial or non-complete meandering	10.39/5.70	5.67/1.35
Braided	6.50/1.79	5.58/1.85
Restricted meandering	5.11/1.10	3.41/0.71
Channel with lateral bars	2.43/0.54	1.18/0.24
Channel with medial bars	1.92/0.64	1.01/0.05

Equally as the ratio of I_o/I also defines the sinuosity of the channel (ratio of channel length to valley length), then for a given valley slope, straight channels have steeper gradients than meandering ones (fig.7.4). The critical ratios of I_o/I for different channel forms are listed in table 7.2.

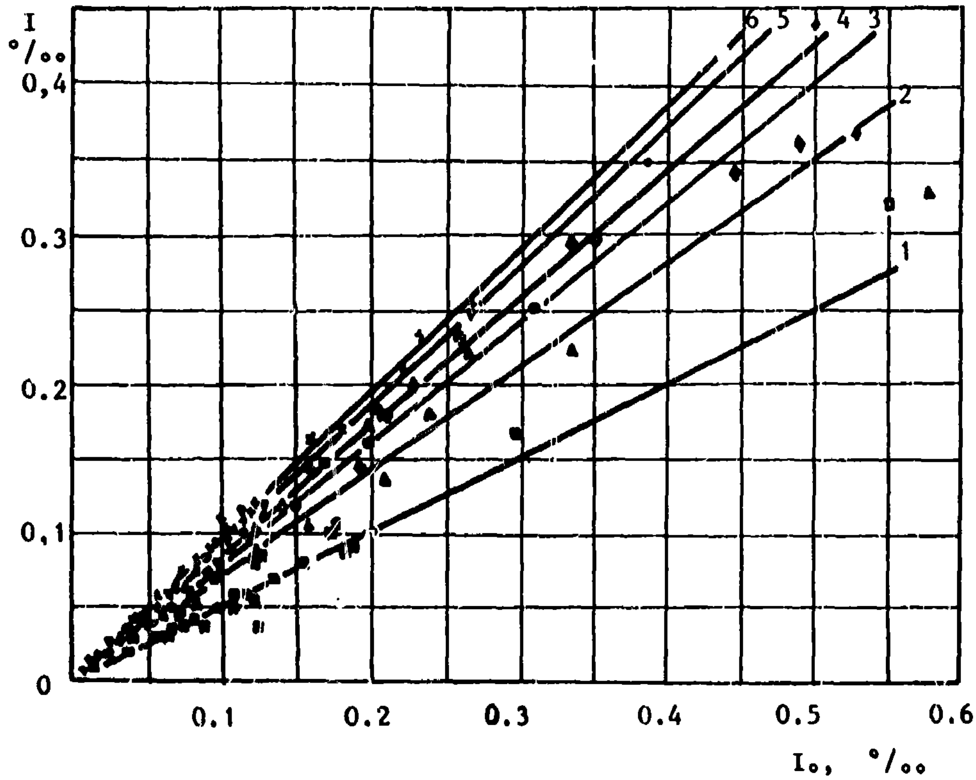


Fig.7.4 Effect of channel slope (I) and valley slope (I_o) on channel form. Symbols are given in Fig. 7.3 (Kondratiev et al., 1982).

TABLE 7.2 Ratio of valley slope to channel slope for different types of channel pattern (N.B. also channel sinuosity)

Type of channel pattern	Mean / Standard
	I_o/I
Free meandering	2.00/0.22
Partial or non-complete meandering	1.41/0.02
Braided	1.22/0.09
Restricted meandering	1.16/0.06
Channel with lateral bars	1.07/0.04
Channel with medial bars	1.03/0.03

From the data relations have been obtained between I_o/I and B_o/B and between I_o/I and B_a/B:

$$I_o/I = 0.05\ (B_o/B) + 0.95 \qquad 7.1$$

$$I_o/I = 0.04\ (B_a/B)^{1.4} + 0.96 \qquad 7.2$$

which enable channel slope to be determined given values of the other parameters.

The products of the two sets of ratios distinguish between different types of channel pattern:

$$(I_o/I)\ (B_o/B) = A \qquad 7.3$$

$$(I_o/I)\ (B_a/B) = A_1 \qquad 7.4$$

as shown in table 7.3.

TABLE 7.3 Mean threshold values of coefficient A for different types of channel pattern.

Type of channel pattern	Mean / Standard
	Criterion A
Free meandering	36.60/11.50
Partial or non-complete meandering	14.65/4.01
Braided	7.93/2.43
Restricted meandering	5.93/1.34
Channel with lateral bars	2.60/0.59
Channel with medial bars	1.97/0.63

A graph of $A = f(Q_{max})$ (fig. 7.5) can also be used to determine channel form, where Q_{max} is the mean annual flood, m^3/s.

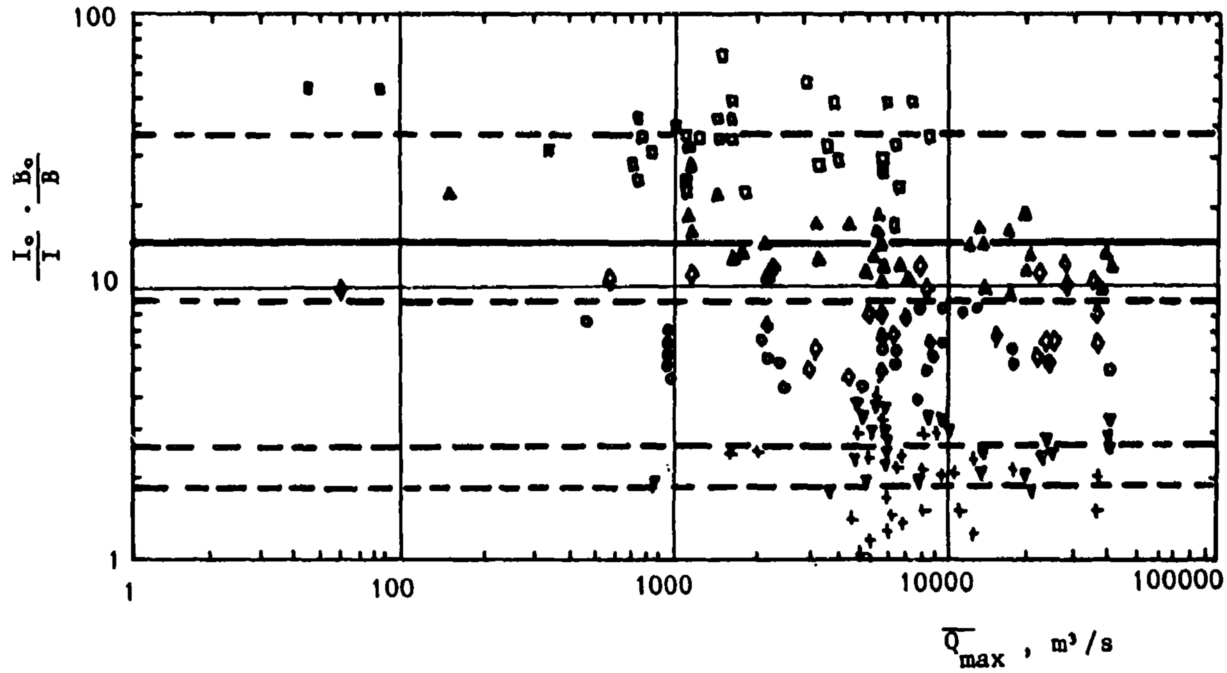


Fig. 7.5 Relation between $(I_o/I)(B_o/B)$ = (A) and mean annual flood (Q_{max}) for different channel patterns. Symbols as for Fig. 7.4. (From Kondratiev et al., 1982).

To predict channel pattern when the flood plain is narrowed by levees, equations (7.3) and (7.4) may be used in addition to the following relations: $I_o/I = f(B_o/B)$; $I_o/I = f(B_a)$; $I_o = f(I)$; $B_o = f(B)$; $A = f(Q_{max})$; $B_a = f(B)$.

Water development projects can either have no effect on channel pattern or the natural pattern can be changed by levee construction, dam operation, etc. In the first case it is necessary to ensure that the project value of coefficient A (A_{pr}) equals the natural value (A_n). The project value can be determined by equations 7.1 and 7.2 and fig. 7.5. Where change is expected, a chosen channel pattern enables a value of A_{pr} to be specified from table 7.3 and fig. 7.5. Subsequently values of I_o/I, B_o/B, B/B are selected which enable the design value of coefficient A_{pr} to be achieved, using figs. 7.3 and 7.4 and table 7.1.

8. PREDICTING EROSION AND DEPOSITION WITH NUMERICAL MODELS

The prediction of erosion and deposition resulting from the construction and operation of river engineering works is of considerable scientific and practical importance. Although much research has been carried out to develop models for predicting scour downstream from dams, there are major discrepencies between the results produced by the different models.

Prendes (1984), as part of the feasibility study for the Chapeton reservoir on the Parana River, demonstrated how it was difficult to predict some morphological changes downstream from the dam and how the results varied, depending on the method used. Accurate prediction of scour downstream from the dam was critical with regard to navigation, stability of the hydro-electric power station, river regulation and, in particular, the safety of a tunnel, located approximately 30km downstream from the Chapeton Dam, linking the cities of Santa Fe and Parana.

Under natural conditions mean annual discharge of the Parana River in this reach is 16 000 m^3/s, the sediment discharge is 3320 kg/s and the silt concentration is 208 g/m^3. The grain size composition of the bed material downstream from the dam site varies slightly, from 0.123 to 1.00 mm, with an average diameter of 0.30mm while the calibre of the suspended sediment ranged from 0.001 to 0.25mm with an average diameter of 0.02mm (fig.8.1).

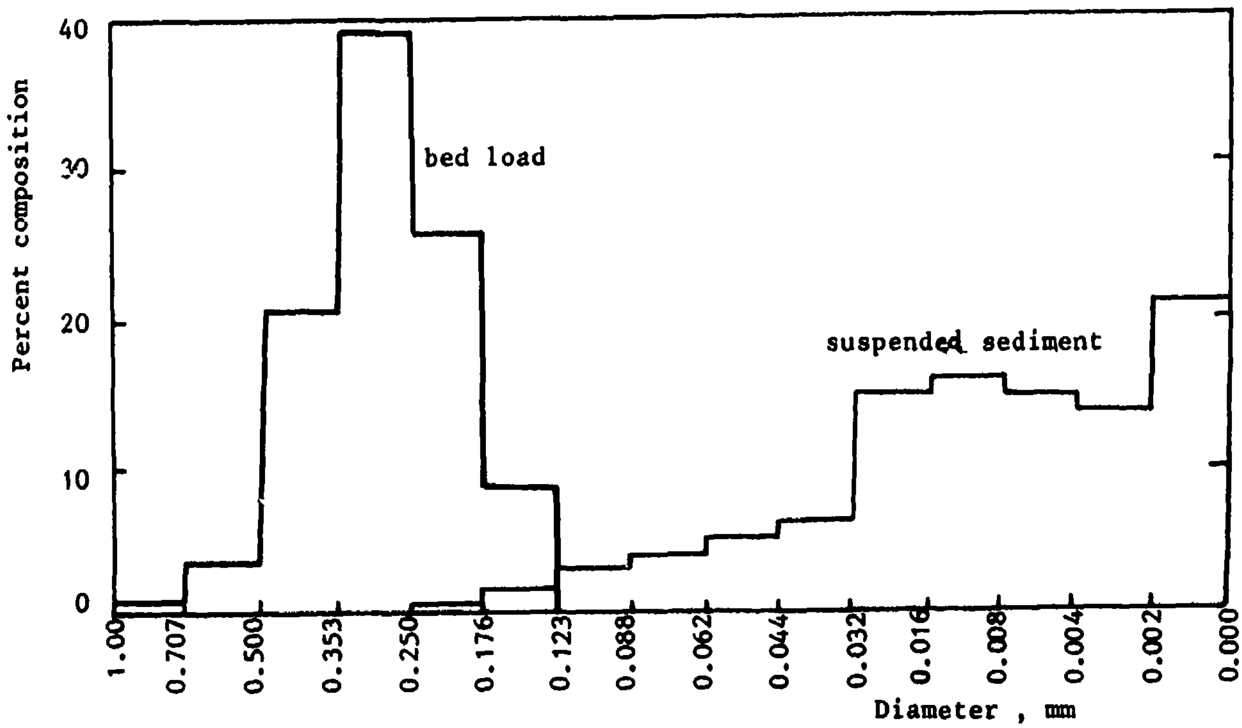


Fig.8.1 Natural suspended and bed load grain size distributions downstream from proposed Chapeton reservoir river Parana (after Prendes, 1984).

In order to determine the response of the river to regulation over a 55km long reach downstream from the dam, mathematical modelling procedures were used, incorporating in turn the formulas of Shamov, Rossinsky, Engelund, Toffaleti and Meyer-Peter for sediment routing. The predicted changes in bed elevation 25 years after the operation of the Chapeton Dam are presented on fig. 8.2, indicating considerable discrepancy between the results.

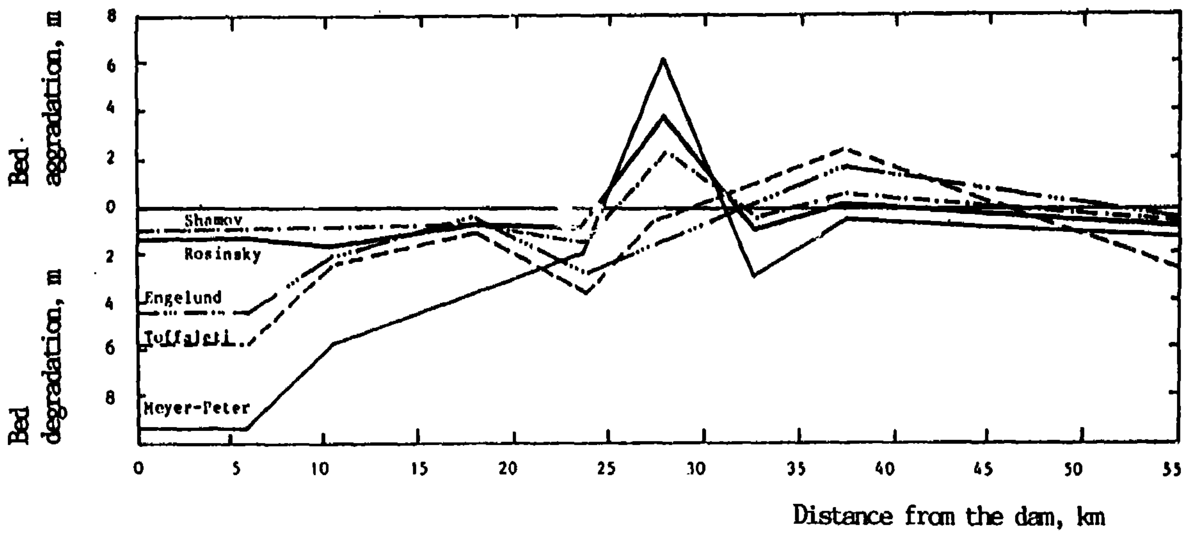


Fig.8.2 Predicted scour and fill downstream from Chapeton reservoir, river Parana, 25 years after dam closure using various sediment transport equations (after Prendes, 1984).

Shamov's and Rosinsky's equations gave comparable results, as did those of Engelund and Toffaleti. The predicted stage-discharge curves downstream from the dam are compared with the current relation in fig.8.3. The largest reduction in water level was predicted by Meyer-Peter's formula.

Obviously the problem is to determine which approach is most accurate and under what circumstances. This can be achieved only by comparing observed and predicted channel changes and by developing improved sediment transport equations.

Many methods have been proposed for the computation of degradation and aggradation downstream from dams and hydro-electric power stations. The general character of these models are similar, differing only in the form of the constituent flow resistance and sediment transport equations. Essentially they are two dimensional models enabling downstream changes in depth to be predicted. Models for predicting width and plan shape changes await further development.

In order to model channel changes it is necessary to specify the future pattern of flows (long term, seasonal, weekly, daily), the present morphology, hydraulic and sedimentary characteristics of the channel, critical threshold velocities for bed material transport, transport capacity of bed and suspended loads under varying flow conditions, the geometry and dynamic nature of any bedforms and methods for predicting armouring and the development of local scour hollows.

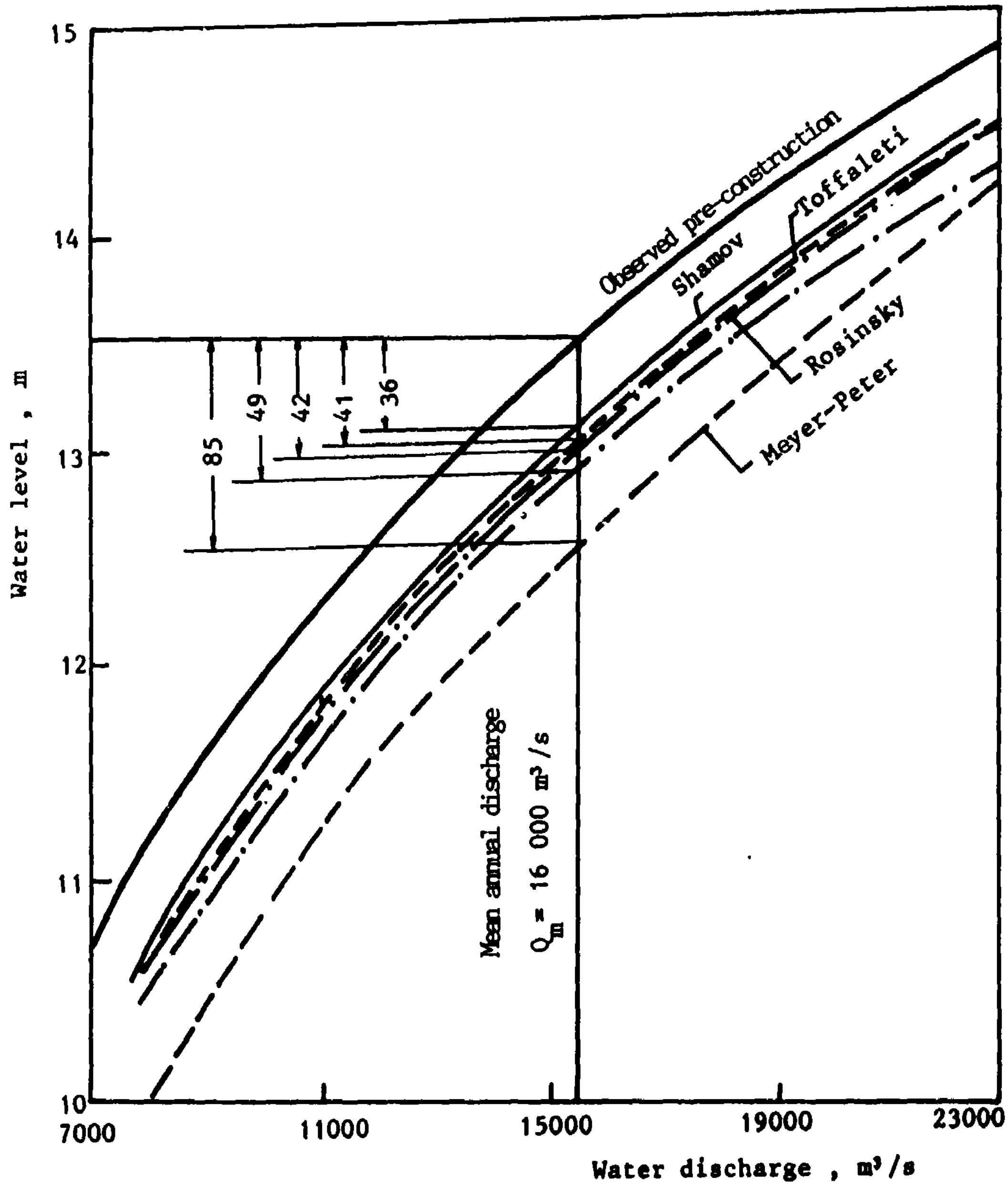


Fig.8.3 Predicted stage/discharge curves for river Parana downstream from site of Chapeton reservoir 25 years after dam closure using various sediment transport equations and current gauged curve (after Prendes, 1984).

The basic equations for computing scour and fill include (Recomendatsii 1981):

momentum equation:

$$\frac{1}{g}\frac{\partial v}{\partial t} + \frac{\partial}{\partial x}\left[\frac{v^2}{2g} + z + h\right] + \frac{\rho_s v q_s}{\rho_w g F} = -i_f, \qquad 8.1$$

water continuity equation:

$$\frac{\partial F(1-S)}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad 8.2$$

sediment mass balance equation:

$$\frac{\partial FS}{\partial t} + \frac{\partial Q_s}{\partial x} = q_s \qquad 8.3$$

bed elevation equation:

$$\rho_s q_s = \rho_{gr} \left(-B \frac{\partial z}{\partial t} + h \frac{\partial B}{\partial t} \right) \qquad 8.4$$

where: z - vertical distance of bed above zero datum

h - mean flow depth

ρ_w, ρ_s, ρ_{gr} - density of water, material in sediment load and bed material

q_s - sediment discharge (inflow or outflow) per unit flow length

F - cross sectional area

i_f - friction slope

v - mean velocity

S - mean sediment concentration

Q - discharge

Q_s - sediment discharge per unit time through cross section

B - water surface width

h_{ed} - bankfull mean depth

α - coefficient which accounts for non-uniform velocity distribution

When solving this system of equations the initial geometrical and hydraulic characteristics of the channel have to be specified at t = o, for several sections along the study reach.

Discharge hydrographs and their associated sediment loads provide the boundary conditions at the first site (x=O). The lower boundary to the study reach is defined by the section downstream from which stability is maintained. However the location of this control section can be modified during the course of the computations.

For rapidly varying unsteady flow the system of controlling equations (based on eqs.8.1-4) including flow resistance and sediment transport equations, can readily be solved by computer using numerical integration methods.

If flows are gradually varying then the equations can be simplified by excluding terms which account for the unsteadiness of the flow (steady state conditions apply for each time interval Δt_j).

The simplified system is based on the following equations:

momentum equation:

$$\frac{\partial}{\partial x}\left(\frac{\alpha v^2}{2g} + z + h\right) = i_j, \qquad 8.5$$

water continuity equation:

$$Q = \text{const.}, \qquad 8.6$$

equation of sediment mass balance and bed scour and fill:

$$\frac{\rho_s \partial Q_s}{\rho_{gr} \partial x} = - B\frac{\partial z}{\partial t} + h_{ed}\frac{\partial B}{\partial t}. \qquad 8.7$$

On practical grounds it is feasible to compute changes in bed elevation using finite difference methods (balance method) (eq.8.8):

$$\frac{\rho_s \ \Delta Q_s}{\rho_{gr} \ \Delta x} = - B\frac{\Delta z}{\Delta t} + h_{ed} \ \frac{\Delta B}{\Delta t} \qquad 8.8$$

In the case of negligible bank erosion ($\frac{\Delta B}{\Delta t} \cong 0$) this gives:

$$\frac{\rho_s \ \Delta Q_s}{\rho_{gr} \ \Delta x} = -B\frac{\Delta z}{\Delta t} \qquad 8.9$$

Total sediment discharge Q_s should be determined as a sum of the bed and suspended load.

For the computation of general erosion, the sediment balance equation is usually written as follows:

$$\frac{\rho_{gr}}{\rho_s} V_{gr} = Q_s \Delta t \qquad 8.10$$

where V_{gr} is the volume of bed material eroded during time interval Δt.

The application of finite-difference approaches requires the division of the reach into sections which have similar hydraulic and geometric properties, thereby enabling the reach to be defined by their average values. The input hydrograph is divided into time increments during which the water and sediment are routed through each channel reach for every time step. After computing the mean bed elevation in a reach for time Δt_j, and the corresponding change in the free water surface, it is possible to compute the change in bed elevation in the next time interval Δt_{j+1} and so on.

Eventually degradation reduces the slope and increases bed material size and, thereby, the transport capacity of the river declines. Consequently erosion will cease.

The input load, as well as the computed sediment discharge downstream, should be evaluated as the sum of the suspended and bed load discharges. To achieve this it is necessary to determine the nature of sediment transport processes at discharges in excess of critical threshold values.

Observations of sediment transport (sliding, rolling, saltation, suspension) may be based on the comparison of the vertical velocity component (v) and the fall velocity of the sediment (ω). Numerous measurements in rivers and flumes (Recomendatsii...,1981; Recomendatsi...,1983; Uchet...,1985) have resulted in the following relations between the shear velocity $u_* = \sqrt{gHI}$ and various measures of the vertical velocity component:

- vertical component ($\bar{v}_{av.v}$), averaged over time and flow depth;
- maximum vertical component ($v_{max.av.v}$), averaged over depth;
- maximum possible vertical component ($v_{max.}$) in the zone of (0.15 - 0.40)H from the bed.

$$\begin{aligned} \bar{v}_{av.v} &= 0.41\,u_* \\ v_{max.av.v} &= 1.28\,u_* \\ v_{max.} &= 1.75\,u_* \end{aligned} \qquad 8.11$$

The nature of the sediment transport process can be determined by comparing the fall velocity with a measure of the vertical velocity component as follows:

$\omega > v_{max.}$	- rolling and sliding
$v_{max.} \geq \omega \geq v_{max.av.v}$	- saltation near the bed
$\bar{v}_{av.v} \leq \omega \leq v_{max.av.v}$	- saltating particles reach mid flow depth
$\omega < \bar{v}_{av.v}$	- sediment in suspension throughout whole flow depth.

Evaluation of the type of sediment transport process may also be made empirically by analysing the grain size composition of the suspended sediment and the bedload (see fig. 8.1 for example).

Bed load discharge may be computed by the formulas of Shamov, Rosinski, Meyer-Peter, Einstein-Brown, Schoklitsch, Engelund-Hansen, Toffaleti, etc. on the basis of bed material size and local hydraulic conditions.

Bed load discharge, involving dune migration with Froude numbers in the range 0.1-1.5 can be determined using the formula developed by Snischenko and Kopaliani (1978):

$$q_s = 0.011\, h_d \frac{\bar{v}^4}{(\sqrt{gH})^3} \qquad (m^3/s/m) \qquad 8.12$$

where:

q_s - bed load discharge/unit width $m^3/s/m$
h_d - dune height in m
h_d = 0.25 H at H < 1m
h_d = 0.20 + 0.1 H at H $\geq$ 1m
$\bar{v}$ - mean velocity in the vertical, m/s
H - flow depth, m

Suspended load discharge may be computed by Karaushev's formula (1977):

$$Q_s = 0.00056\, QNh^2 \frac{\bar{v}^2}{gH} \theta. \quad kg\ s^{-1} \qquad 8.13$$

where:

$$N = \frac{(0.7C + 6)C}{g} \qquad 8.14$$

$$h^2 = \frac{0.53C - 4.1}{C-2} \qquad 8.15$$

Q_s - suspended load discharge $kg\ s^{-1}$
C - Chezy coefficient
θ - Hydraulic parameter determined from auxiliary graphs (fig.8.4)
H - flow depth, m
w - fall velocity, m/s
$\bar{V}$ - mean flow velocity, m/s
Q - discharge, m^3/s

Where field data on natural suspended loads are available, it is possible to determine the sediment discharge Q_s using a method devised by Rosinski and Kuzmin (1964). This is based on the dependence of the suspended sediment concentration (S) on a parameter characterising the flow hydraulics (v^3/H).

Two curves are plotted:

the upper curve:

$$S = 23.7\, \rho_w \frac{v^3}{gH\bar{w}} \times 10^{-5}\ g\ m^{-3} \qquad 8.16$$

the lower curve:

$$S = 4.75 \rho_w \frac{v^3}{ghH\bar{w}} \times 10^{-5}\ g\ m^{-3} \qquad 8.17$$

where: v - flow velocity, m/s
H - flow depth, m
ρ_w - density of water, g/m^3
$\bar{w}$ - mean fall velocity, m/s

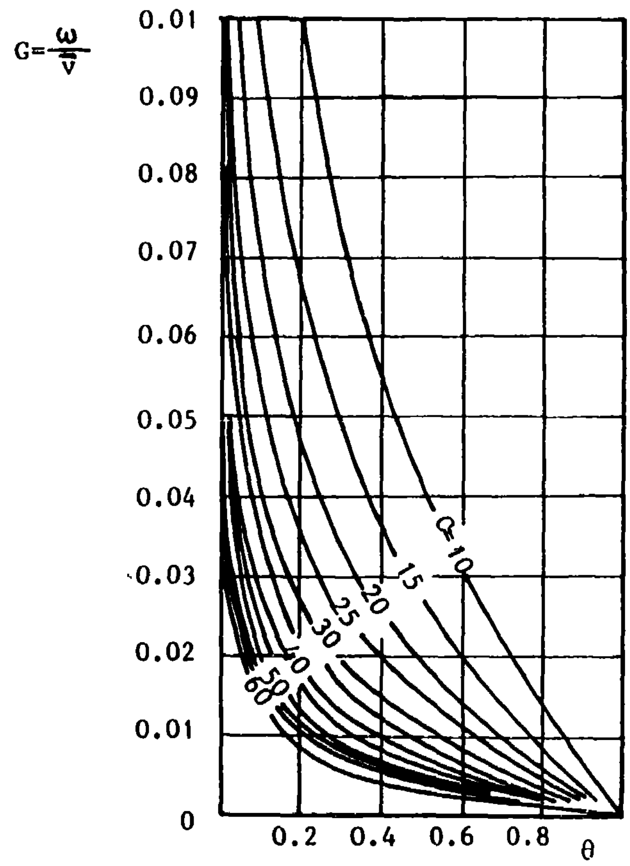


Fig.8.4 Graph for determination of hydraulic parameter θ (after Karaushev, 1977).

The final requirement is the determination of the equilibrium condition. On rivers with fine sand beds equilibrium is achieved as a result of a decrease in the water surface slope. In gravel-bed rivers the channel stabilizes due to slope reduction combined with bed armouring.

The maximum channel depth (H_{max}), when erosion ceases in uniform material, can be determined from:

$$H_{max} = \frac{q_i}{b_i u_o} \quad m \qquad 8.18$$

where:

q_i - discharge in part of channel cross section b_i (m) wide ($m^3 s^{-1}$)

u_o - non-eroding velocity of flow ($m\ s^{-1}$)

Determination of the maximum channel depth when the bed material is non-uniform, requires allowance to be made for the development of an armoured surface on the bed of the channel (eq.8.19)

$$H_{max} = \frac{Q}{Bu_{fin}} \text{ m} \qquad 8.19$$

where u_{fin} is the flow velocity at which armouring occurs

Q - discharge m^3s^{-1}

B - channel width m

There are many formulas, often presented graphically, for determining of u_o and u_{fin}. For example, the non-eroding velocity u_o for uniform non-cohesive material may be determined from figure 8.5. The flow velocity u_{fin} at which armouring of the river bed occurs in heterogeneous non-cohesive material can be determined from figure 8.6, where k_Q (= 100-p/100) is the relative content of coarse particles in the bed material. The value of p is obtained graphically by plotting a grain size/probability curve and obtaining the largest value of p (% finer) at which there is a significant change in curvature. In the example in Fig. 8.7, k_c = 100-80/100 = 0.2 If no distinct change of curvature is observed, it is recommended that p is set equal to 10 percent.

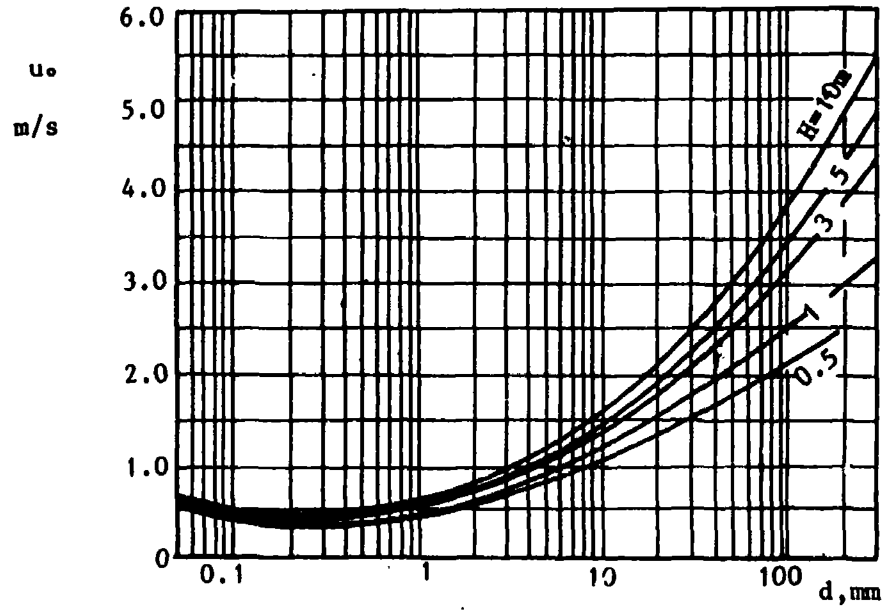


Fig.8.5 Non-eroding velocity for homogeneous non-cohesive bed material (Rekomendatsii..., 1981b). (H = local flow depth, d = grain diameter).

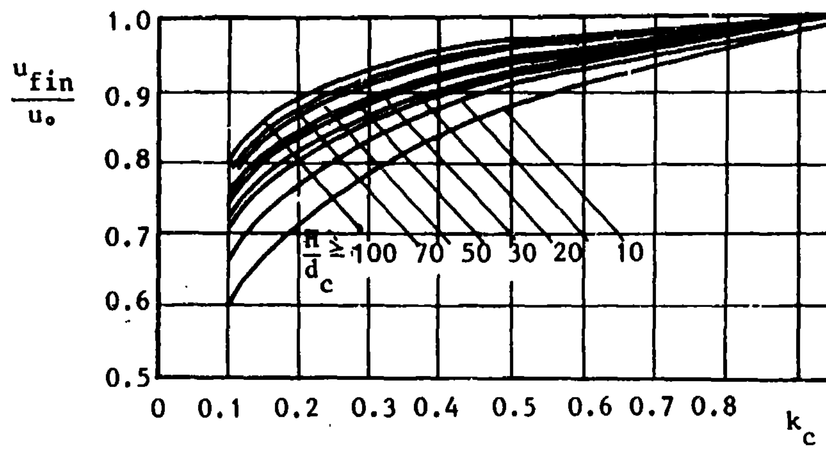


Fig.8.6 Non-eroding velocity u_{fin} at which armouring of the river bed occurs (Rekomendatsii..., 1981b). (H = local flow depth, d_c = mean diameter of coarse particles).

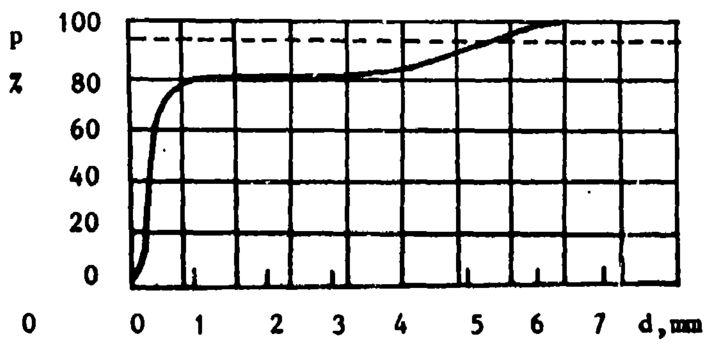


Fig.8.7 Grain size distribution of bed material.

Comparisons between the observed and the predicted response of a river to the construction and operation of engineering works indicates the need for further development of modelling procedures. This requires research into the physical processes controlling erosion and deposition in alluvial channels.

9. CONCLUSION

This report presents numerous examples of the effect of dam construction, reduction in base levels, dredging bed material and regulating and training works on channel stability and outlines empirical and mathematical procedures for predicting channel response to such changes.

The examples, based on published and unpublished reports from many countries, indicate the dramatic effects that erosion and deposition can have on the stability of bridges and hydraulic structures, as well as their adverse effect on groundwater systems. Future development of river basins for water supply, flood control, etc. is likely to exacerbate the problem.

Typically the expense of repairing a collapsed structure is often greater than the cost of preventive measures. Often this results from ignorance of the degree and extent of erosional and depositional activity occasioned by changes in the flow regime and patterns of sediment run-off. Rather than attempting to predict instability problems and design appropriate protective measures, the problem is usually ignored until remedial measures are required. The cost of rectifying a problem can be substantial and it would be more efficient and timely if potential problems could be identified at the design stage. Often protective measures are required only for a short period of time, particularly during flood events.

Prediction of natural erosion and deposition is a pre-requisite for modelling man-induced changes. Unfortunately, existing methods for predicting erosional and depositional activity in response to river engineering works and land use changes give variable results due to lack of general physically based equations for modelling flow resistance, sediment transport and armouring processes.

To aid the development of better modelling procedures long-term field investigations of unstable channels, both natural and man induced, need to be carried out. This would entail the systematic collection of information on discharge regime, sediment supply and channel morphology and sedimentology. Field and laboratory studies, in combination with theoretical investigations will enable the assessment of existing models and the development of new approaches for modelling scour and fill in alluvial channels. IHP has a key role to play in this respect through the coordination of theoretical, field and laboratory studies in different countries and the organization of meetings to discuss research findings. This would significantly aid the development of general and reliable modelling procedures for predicting channel response to man induced changes.

10. REFERENCES

Agostini, R., Bizzari, A. and Masetti, M., 1981. Flexible structures in river and stream training works, report, Officine Macaferri Bologna, Italy.

Altunin, S.T., 1958. Zaile vodohranilisht i kazmyv v rusla v nijnem biefe plotin. Ruslovye protzesy (Silting of reservoirs and river bed erosion downstream from a dam. River bed processes), Izd.AN SSSR, Moskva.

Babinski, Z., 1982. Procesy korytowe Wisly ponizej zapory wodhej we, Wloclawku Polska Akademia Nauk.

Bauer, F., 1963. Gutachten über die Entwicklung des Geschiebehaushaltes der Donau zwischen Kelheim und Isarmündung.

Bauer, F., 1965. Der Geschiebehaushalt der bayrischen Donau im Wandel wasserbaulicher Massnahmen. Die Wasserwirtschaft, Heft 4 und 5.

Bauer, F., 1979. Das flussmorphologische Verhalten des bayerischen Lech, Schrittenreihe Bayer. Landesamt für Wasserwirtschaft, Heft 9, München.

Bauer, F. and Burz, I., 1968. Der Einfluss der Feststofführung alpiner Gewässer auf die Stauraumverlandung und Flussbetteintiefung. Die Wasserwirtschaft Heft 4, Stuttgart.

Bensing, W., 1966. Gewässerkundliche Probleme beim Ausbau des Oberrheins. Deutsche Gewässerkundliche Mitteilungen, 10, 7g, H.4.

Bondar, K. and State, I., 1977. The Danube River Contribution with Water and Silt to the Black Sea. Cercetary Marine, 10, Bucarest.

Borshuk, O.A. and Chalov, R.S., 1973. Downcutting on the Lena River Channel. Soviet Hydrology, Selected Papers, 5.

Brune, I.M., 1953. Trap efficiency of reservoirs, Trans. Amer. Geophys. Union, 34, 3.

Coldwell, A.E., 1947. Effects of sediment on design and operation of dams and reservoirs. Federal Inter-Agency Sedimentation Conference, Denver, Colorado.

Coleman, J.M., 1969. Brahmaputra River, Channel Processes and Sedimentation. Sedimentary Geology, 3.

Cory, H.T., 1913. Irrigation and river control in the Colorado River delta. American Society of Civil Engineers Transactions, 76.

Daniels, R.B., 1960. Entrenchment of the Willow Creek drainage ditch Harrison County, Iowa, Am.J.Sci., 225.

De Greer, S., 1910. Klarälfveus serpentin lopp och flodplan. Sveriges Geologiska Undersökning, C, (236).

Die Erosion der Rheinsohle zwischen Oppenheim und Koblenz. Deutsche Gewässerkundliche Mitteilungen, 5, 1960.

Emmett, W.W., 1974. Channel aggradation in Western United States as indicated by observations at vigil network sites. Zeitschrift für Geomorphologie, 21.

Eschweiler, W., 1952. Die Wasserspiegel- und Sohlensenkungen am Niederrhein. Bes. Mitt. Zum Deutschen Gewässerkundlichen Jahrbuch, 7, Koblenz.

Felkel, K., 1969. Die Erosion des Oberrheins zwischen Basel und Karlsruhe. Das Gas- und Wasserfach, 30.

Gage, M., 1970. The tempo of geomorphic change. Journal of Geology, v. 78.

Galay, V.I., 1983. Causes of River Bed Degradation. Water Resources Research, 19, (5).

Galay, V.I., Halliday, R. and Pentland, R., 1983. Degradation of the South Saskatchewan River below Gardiner Dam. Paper presented at the Annual Conference, Can. Soc. Civ. Eng., Edmonton, Alberta.

Glazik, G., 1964. Veränderungen der Wasserspiegel- und Sohlenlage der Elbe, Wasserwirtschaft-Wassertechnik, 11.

Gupta, P.O., Gupta, S.N. and Saxena, P.M., 1967. Effects of flood hydrograph on the regime of alluvial and boulder stage river above and below control structures. 12 Congress Int. Assoc. Hydraul. Res. Fort Collins, Colorado.

Hammad, Y., 1982. River bed degradation after closure of dams. J. Hydraul. Div. Amer. Soc. Civ. Eng., 98 (4).

Hathaway, G.A., 1948. Observations on channel changes, degradation and scour below dams. 2nd International Association Hydraulic Structures Research Meeting, Stockholm.

Hickin, E.I. and Nanson, G.JC., 1975. The character of channel migration on the Beaton River, northwest British Columbia, Geological Society of America Bulletin, 86.

Karasev, I.F., 1975. Ruslovye protzesy pri perebroska stoka (River bed processes at run-off re-distribution). Gidrometeoizdat., Leningrad.

Karaushev, A.V., 1965. Metod rascheta osajdenia nanosov v vodohranilishtah (Method of computation of sediment deposition in reservoirs). Trudy GGI, vyp. 124, Leningrad.

Karaushev, A.V., 1966. Teoria i metod rascheta zailenia malyh vodohranilischt i prudov (Theory and method of computation of silting in small reservoirs and ponds). Trudy GGI, vyp. 132, Leningrad.

Karaushev, A.V., 1977. Teoria i metody rascheta rechnih nanosov (Theory and methods for river sediments computation), Gidrometeoizdat, 270p, Leningrad.

Kesely, H.M., 1977. Landsliding, channel changes, sediment yield and land use in the Van Duzen river basin, north coastal California, 1941-1975, University of California at Santa Cruz.

Kondratiev, N.K., Popov, I.V. and Snishenko, B.F., 1982. Osnovy gidromorfologicheskoi teorii ruslovogo protzesa (Principles of the hydromorphological theory of river bed deformations). Gidrometeoizdat, 271 p, Leningrad.

Kopaliani, Z.D., 1984. O sootnoshenii rashodov donnyh i vzveshennyh nanosov v rekah (On relations between bedload and suspended sediment discharge in rivers). Abstracts of papers of the second All-Union Conference "Dynamics of Rivers, Reservoirs and Estuaries", vol. I, Moscow, p.173-176.

LaMarche, V.C., 1966. An 800-year history of stream erosion as indicated by botanical evidence. U.S. Geological Survey Professional Paper, 500-D.

Lane, E.W., 1955. The importance of fluvial morphology in hydraulic engineering. Proceedings American Society of Civil Engineers, v. 81.

Leopold, L.B., Emmett, W.W. and Myrick, R.M., 1966. Channel and hillscope processes in a semiarid area, New Mexico, U.S. Geological Survey Professional Paper, 352-G.

Levi, I.I., 1938. Otstoiniki i promyvnye ustroistva proektirovanie, teoria i raschet (Clarification beds and filtration equipment design, theory and computation), ONTI.

Li, B., Hua, Z., Fan, Z. and Chen, S., 1980. Changes of the river channel downstream of Sanmexia Reservoir during silt retention period. Paper presented at the Intern. Symposium on River Sedimentation. Chin Soc. of Hydraul. Eng., Beijing, China.

Lisitzina, K.N., 1969. Raschet zailenia prudov (Computation of pond silting). Trudy GGI, vyp.175, Leningrad.

Lopatin, G.V., 1961. Izuchenie protzesa zailenia malyh vodoemov v usloviah Kurskoi oblasti (Study of silting process in small reservoirs in Kurskoi area), Trudy Labor. Ozero.edenia AN SSR, tom XIII.

Lustig L.K., 1965. Sediment yield of the Castaic watershed. Los Angeles, California, U.S. Geological Survey Professional Paper, Paper 422-F.

Lvovitch, M.I., 1974. Mirovye vodnye resursy i ih budushte (World water resources and their future). Izd. Mysl., Moskva.

Mandych, A.F. and Chalov, R.S., 1970. Trend of present-day erosion-accumulation processes in the lower reaches of rivers in the Western Transcaucasus. Soviet Hydrology, Selected Papers, 1.

Methods of computing sedimentation in Lakes and Reservoirs, 1985. UNESCO, Paris.

Mostkov, M.A., 1950. O raschete zailenia vodohranilisht (On the computation of reservoir silting). Trudy TbIIJT, No. 23.

Nenov, I., 1985. Ruslovi deformatzii v dolnia uchastak na iazovirite v Bulgaria (River bed deformations downstream from dams in Bulgaria). Dokladi na instituta poHydr. i Meteor., Sofia.

Northwest Hydraulic Consultants Ltd., 1976, Saskatchewan River delta study, report to Duck Unltd., Winnipeg, Manitoba.

O'Loughlin, C.L., 1969. Streambed investigations in a small mountain catchment. New Zealand Journal of Geology and Geophysics, 12. (4).

Orth, F., 1934. Die Verlandung von Staubecken. Die Bautechnik, H. 26.

Pau Ching-shen, Shin Shao-chuan and Tuan Wen Chung, 1978. A study of the channel development after the completion of artificial cut-offs in the middle Yang-Tse River, Sci. Sinica, XXI (6).

Pechinov, D., 1982. Izmemenia granulometricheskogosostava vzveshennyh nanosov r. Dunai pod vlianiem Jeleznyh vorot I (Change of grain-size composition of suspended sediments under the influence of the Iron Gate I). XI konferentzia pridunaiskih stran po gidrologichaskih prognozom, Bucharest.

Pechinov, D., 1984. Ruslovi deformatzii na r. Dunav v dolnia uchastak na Jelezni vrata I (River bed deformations of the Danube River downstream from Iron Gate I). Seminar po proekt A.3.8 IHP, Sofia.

Pichi, K., 1958. Die Erosion der Rheinsohle zwischen Karlsruhe und Oppenheim. Die Wasserwirtschaft, 11 (12), 487.

Poliakov, B.V., 1935. Zailenia vodohranilisht zavoljia (Reservoir silting in Zavoljkoi area). Nijnevolgoproekt, vyp.V, Moskva--Leningrad.

Prendes, H., 1984. A mathematical model with movable bed and its application to the Parana River. Agua y Energia Electica, Argentina.

Raynov, Scht., Nenov, I., Vassilev, S. and Pechinov, D., 1979. Granulometrische Zusammensetzung der Bodenablagerungen der Donau. X. Konferenz der Donauländer über hydrologische Vorhersagen, Wien.

Raynov, Scht., Nenov, I. and Pechinov, D., 1981. Die Verteilung der Schwebstoffbelastung des Donauquerschnitts bei Lom. International Arbeitsgemeinschaft Donauforschung. Arbeitstagung XXII, Basel.

Raynov, Scht., Ninov, I., Pechinov, D., Gergov, G. and Nenov, I., 1977. Izmenenie nanosov r. Dunai pod vlianiem gidrotechnicheskogo kompleksa Jeleznye voroty I (Change of sediments in the Danube River under the influence of hydrotechnical complex Iron Gate I). IX konferentzia pridunaiskih stran po gidrologicheskih prognozam, Budapest.

Recomendatsii po raschetu transformatsii rusla v niznih bijefah gidrouzlov, 1981 (Recommendations for the computation of channel transformations in the lower pools of hydropower plants), VNIIG, Leningrad, 96p.

Recomendatsii po razmeshcheniu i proektirovaniu rasseivaiushchih vypuskov stochnyh vod, 1981a (Recommendations on location and methods for discharging waste water). Moscow, Stroiizdat, 224 p.

Recomendatsii po raschetu mestnyh razmyvov rusel slozhennyh iz neskolkih gruntov so krepleniami srednonapornyh vodoslivnykh plotin, 1981b (Recommendations for the computation of local erosion in channel composed of non-rocky material downstream from the apron of mid-pressure dams). VNIIG, Leningrad, 37 p.

Rekomendatsii po uchetu deformatsiy rechnih rusel pri proektirovanii inzhenernyh sooruzheniy na rekah zony BAM, 1983 (Recommendations on the inventory of river bed deformations in case of hydraulic projects in the rivers of the Baikal-Amur railway area). Leningrad, Gidrometeoizdat, 71 p.

Richter, W., 1964. Zur Flussbettumbildung unterhalb von Flusskraftwerken. Diss, TU Berlin.

Ritter, J.R., 1968. Changes in the channel morphology of Trinity River and eight tributaries. California, 1961-65. U.S. Geological Survey Open-File report.

Romashin, V.V., 1968. Typy ruslovogo protsesa v sviasi s opredeliaushchimi faktorami (Types of the channel processes in connection with the factors determining these processes). Trudy GGI, vol.155. Leningrad, Gidrometeoizdat, p.56-63.

Rosinsky, K.I. and Kuzmin, I.A., 1964. Balansovy metod rascheta deformatzii dna potoka (Balance method for computing river bed deformations). Trans.Gidroproekt, vol.12.

Rukovodstvo po opredeleniu dopustimyh nerazmyvaiushchih skorostei vodnogo potaka dlia razlichnyh gruntov pri raschete kanalov, 1981 (Guide on the determination of admissible non-eroding flow velocities for various bed material for canal projects) Minvodkhoz SSSR, Moscow, 57 p.

Scheurmann, K., 1973. Die Pupplinger- und Ascholdinger Au in flussmorphologischer Ansicht. Bayerisches Landesamt für Wasserwirtshaft, Heft 7.

Scheurmann, K., 1975. Flussmorphologische Veränderungen der Unteren Isar und deren Auswirkung auf die Donau, VIII Konferenz der Donauländer über hydrologische Vorhersagen, Regensburg.

Scheurmann, K., Weiss, F.H. and Mangeldort, J. 1980. Die flussmorphologische Entwicklung der Salzach von der Saalachmündung bis Mündung in den Inn. Bayerisches Landesamt für Wasserwirtschaft, 2.

Schmutterer, J., 1952. Welches Ausmass haben die Sohlenänderungen der Donau. Ostereichische Wasserwirtschaft, Heft 8 (9).

Schoklitsch, A., 1914. Uber Schlapkraft und Geschiebebewegung. Leipzig und Berlin.

Sedimentation problems in river basins, 1982, Project 5.3 IHP, UNESCO, Paris.

Sediment control methods d.reservoirs. 1973. J. Hydraul. Div. Proc. Amer. Soc. Civ. Eng., 99 (4).

Serebriakov, A.V., 1970. Ruslovye protsesy na sudohodnih rekah s zaregulirovannym stokam (River bed processes in navigable rivers with regulated run-off). Izd.Transport, Moskva.

Shalash, S., 1982. Effects of sedimentation on the storage capacity of the High Aswan Dam reservoir. Hydrologia 91, Netherlands.

Shalash, S., 1983. Degradation of the river Nile. Water Power and Dam Construction.

Shamov, G.I., 1939. Zailenie vodohranilischt (Reservoir silting). Gidrometeoizdat. Moskva - Leningrad.

Shamov, G.I., 1954. Rechnye nanosy (River sediments). Gidrometeoizdat. Leningrad.

Simons, D.B. and Senturk, F. 1977. Sediment transport technology. Water Resources Publications, Fort Collins, Colorado.

Skrylnikov, V.A., 1961. Raschet zailenia podperthyh biefov gidrouzlov (Computation of silting in backwater reaches of hydraulic structures). Sb. Voprosy energetiki, gidrotehniki i gornogo dela. Tashkent.

Smith, D.G., 1972. Aggradation and channel patterns of Alexandera-North Saskatchewan River, Banff National Park, Alberta, Canada, in Slaymarker, O. and McPherson H.J., eds. Mountain Geomorphology, Vancouver.

Snishchenko, B.F., 1979. Sviaz tipov s formani rechnyh dolin (Relations between channel types and river valley forms). Geomorphologia, Moscow, Nauka, 1, 18-25.

Snishchenko, B.F. and Kopaliani Z.D., 1978. O skorosti dvizenia griad v rekah i laboratornih usloviah (On the velocity of dune movement in rivers and laboratories). Trans.GGI, 252, Leningrad.

Snishchenko, B.F. et al., 1982. Izmenenie gidrologicheskogo ruslogo rejima rek pod vlianiem vyemkah aluvia (podvodnyh karerov) i puti ego prognoza (Change of river bed regime under the influence of alluvium dredging and its prediction). IX konf.Pridunaiskih stran po gidrologicheskim prognozam. Bucharest.

Tingsanchali, T. and Overbeek, H.J. 1980. Monographical study on the effects of sand dredging in the Chao Phraya River, Thailand. AIT, Res.Rep.96.Bangkok.

Tippner, M., 1973. Uber den Umfang der Sohlenerosion in grossen Gewässern. Sonderdruck aus Deutche Gewässerkundliche Mitteilung, 5.

Todd, O.J. and Eliassen, J. 1940. The Yellow River problem. American Society of Civil Engineers Transactions, 105.

Uchet deformatzii rechnyh rusel i beregov vodoemov v zone podvodnyh perehodov magistrlnyh truboprovodov (neftogazoprovodov), 1985 (Inventory of river-bed and shore deformations at the sites of underwater trunk pipelines [oil and gas pipelines]). VSN 163-83. Leningrad, Gidrometeoizdat, 142p.

Ukazania po raschetu zailenia vodohranilisht pri stroitelnom proektirovanii (Instructions for computation of reservoir sedimentation at design stage) Gidrometeoizdat. Leningrad, 1973.

Untersuchungen zur Frage, ob Sohlenerosion des Oberrheins unterhalb der Staustufe, Iffezheim durch Geschiebezugabe, weitere Staustufen oder Grundschwellen verhindert werden kann. Der Bundesminister für Verkehrsabteilung Binnenschiffahrt und Wasserstrassen, Bonn, 1981.

U.S. Bureau of Reclamation, 1945. Tenth annual report on retrogression observations, salinity studies and suspended load investigations, Colorado River basin. Boulder City, Colorado.

U.S. Bureau of Reclamation, 1967. The 1965 sedimentation survey of Augostura Reservoir South Dakota. Denver, Colorado.

Veksler, A.B. and Donenberg, V.M., 1983. Pereformirovanie rusla v nijnih biefah krupnih gidroelektrostantzii (Reformation of river bed downstream from large hydro-power stations). Energoatomizdat. Moskva.

Weiss, F.H., 1983 Report on project A.3.8 IHP at the Seminar in Sofia.

Wetter, S., 1953. Sediment problems in Lake Mead downstream of the Colorado river. Trns.Amer.Geophys.Union, 2.

Williams, G.P. and Guy, H.P., 1973. Erosional and depositional aspect of Hurricane Camille in Virginia. U.S. Geological Survey Professional Paper, 804.

Williams, G.P. and Wolman, M.G., 1984. Downstream Effects of Dams on Alluvial Rivers. U.S. Government printing office, Washington.

Winkley, B.R., 1977. Man-made cutoffs on the lower Mississippi River, Conception, construction and river response, Report. Potamology section, US Army Corps. of Engs., Vicksburg District, Vicksburg, Mississippi.

Zubentz, V.M. and Stankevitch, V.N. 1985. Reformirovanie rek na priplotinnon uchastke gidrologicheskoe stroitelstvo (Reformation of rivers in the reaches near hydrological constructions) kn.3, Moskva.