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4eme CONGRES INTERNATIONAL
ASSOCIATION INTERNATIONALE DE GEOLOGIE DE L'INGENIEUR
INDE—1982

VOLUME VII

THEME 4
Engineering geological problems of natural and man-made lakes
Problèmes de la géologie de l'ingénieur des lacs naturels et artificiels

THEME 5
Engineering geological problems of sea-coast and shelf areas
Les problèmes de la géologie de l'ingénieur des régions côtières et des bancs continentaux

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The Organising Committee of the 4th International Congress of Engineering Geology feels particularly happy to be able to release in time the nine volumes of the Proceedings of the Congress, running to about 3000 pages. The first eight volumes encompass over 270 scientific and technical papers, one-third of which are from India and two-thirds from 32 other countries of the world. The ninth volume contains the Panelists' Reports, the Sessional Reports and the Special lectures planned to be delivered during the Congress. The tenth volume, containing the record of the Technical Sessions and the discussions to be held during the Congress, is proposed to be released at a later date.

The seven Themes as chosen for discussion during the 4th Congress cover a wide range of topics, all of which are of special interest to the present-day developmental activities in the world.

The listed Themes are:
2. Engineering Geological Problems of Tunneling and Excavation of Cavities;
3. Soil and Rock as Construction Material;
4. Engineering Geological Problems of Natural and Man-made Lakes;
5. Engineering Geological Problems of Sea-coast and Shelf Areas;
6. Seismic and Seismo-tectonic Investigations of Engineering projects; and

The themes as listed above have been further sub-classified into 31 sub-themes, so as to bring into their fold several related problems and to focus attention on specific aspects.

Although the Organising Committee feels very much gratified by the overwhelming response to the call for scientific and technical papers for discussion during the 4th Congress, and nearly 300 papers were received from different countries of the world, the major and somewhat perplexing task faced by the Committee was to classify the papers received and to pigeon-hole them into the various Themes and sub-themes. In fulfilling this onerous task, the Committee has done its best in good faith and has offered a classification in the eight volumes of the Proceedings which may be taken as one of the best-fits for the Themes and the sub-themes and not as the only best possible fit. Again, by stretching the ambit of some of the themes, a number of papers, which were of great topical interest and were considered to be of an auxiliary nature as well as related to the Themes posed for discussion, were accommodated, as is particularly the case with Theme 1. In some of the papers, minimal language corrections were made, taking care, however, to retain the style of the authors' presentations to the maximum possible extent.

A very large number of papers were received for Theme 1 and these have been distributed in the first three volumes: the first volume containing all the papers falling "sensu stricto" under Theme 1 while Volumes II and III containing all the papers of an auxiliary or supporting nature and which are, in one way or the other, related to Theme 1. Theme 2 had also more than 50 contributions; therefore, these have been accommodated in two volumes, Volumes IV and V. While the papers for Theme 3 have been included in Volume VI, the papers for Themes 4 and 5, put together, have been incorporated in Volume VII and those of Themes 6 and 7, put together, in Volume VIII. The Organising Committee thanks all the contributors for their generous cooperation. The Committee is grateful to the various National Groups for the trouble they had taken to collect the papers from their respective countries and to forward these to the Organising Committee for its consideration. The Geological Survey of India has been the backbone of all activities relating to the Congress and the work of publication of the Proceedings of the Congress has not been an exception in its contribution. The Organising Committee is under a deep debt of gratitude to the Director General, Geological Survey of India and to the Officers working under him in different geoscientific disciplines, for their generous and unfailing support.

The main burden of the review of all the papers and the editing of some of them was shared by M/s.

New Delhi
12 September 1982

The Organising Committee is obliged to all these geoscientists for their unremitting help and support. M/s. Oxford & IBH Publishing Co., New Delhi, did a commendable job in bringing out all the volumes of the Proceedings of the Congress in very good shape and in good time, despite several odds faced by them. They deserve the sincere thanks of the Organising Committee.

V.S. Krishnaswamy
Chairman, Organising Committee
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Theme 4

ENGINEERING GEOLOGICAL PROBLEMS OF NATURAL AND MAN-MADE LAKES
SILTATION OF RESERVOIRS IN GANGA YAMUNA VALLEY

ENVASEMENT DES RESERVOIRS DE LA VALLEE DE GANGA YAMUNA

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ABSTRACT

There is enormous power and irrigation potential in the rivers of Ganga Yamuna valley in central Himalayas. The potential can be exploited by constructing water resources projects on the perennial rivers; a few of them have already been completed and commissioned. The reservoirs in the valley have presented serious problems of siltation which is attributed to the heavy sediment concentration in the rivers during monsoons. Prototype observations of Ichari reservoir have indicated that total sediment transported by river Tons during 1978-79 was of the order of 29 M. cu.m (million cubic metres) for a river flow of 7892 M. cu.m. Due to heavy sediment concentration the reservoir of Ichari and Maneri dams has been silted up to the crest of spillway within a short span of 2 years.

The catchment area of the rivers comprises of Himalayan slopes which are in general unstable and steep with rocks highly fractured, faulted and folded. The rocks consist of a variety of geological formations ranging in age from lower Paleozoic to Tertiary. The older rocks consist of quartzites, phyllites and slates. The ridges are reported to have been made up of highly weathered dolomites and magnesites subjected to severe temperature variation both diurnal and seasonal. The valley is also reported to have been subjected to frequent seismic disturbances resulting in loosening and breaking up of the rock formation thereby causing frequent landslides. These are largely responsible for heavy sediment concentration in the rivers which results in siltation of reservoirs.
ABSTRAIT

Il existe d'énormes potentiels d'énergie et d'irrigation dans les rivières de la vallée de Ganga Yamuna aux Himalayas du centre. Ces potentiels peuvent être exploités par la construction des projets de ressources en eau sur les rivières interrissables, dont certains ont été déjà achevés et mis en fonctionnement. Ces retenues rencontrent de sérieux problèmes d'envasement du à la forte concentration de sédiment dans les rivières durant les moussons. Des études menées sur la retenue de Ichari ont indiqué que la quantité totale de sédiment transporté par la rivière Toms en 1978-79 était de l'ordre de 29 millions de mètres cubes pour un écoulement de 7992 millions de mètres cubes. À cause de la forte concentration de sédiment, la retenue de Ichari et les barrages de Maneri ont été envahis jusqu'au niveau de l'évacuateur, dans un intervalle de 2 ans.

Le bassin hydrographique est composé de pentes himalayennes qui sont en général instables et abruptes avec des roches brisées, mutulées et plissées. Ces roches sont de diverses formes géologiques, appartenant à l'époque allant de l'ère paléozoiq à l'ère Tertiaire. Les anciennes roches sont composées de quartz, phyllites et ardoises. Les crêtes rocheuses sont constituées par les dolomites et les magnésites altérées par les intempéries et les grandes variations de températures diurnes et saisonnières. La vallée est également sujette à de fréquents mouvements sismiques dus au désarrement et à la brisure des roches causant ainsi de fréquents glissements de terrain. Tout ce changement est du à la forte concentration de sédiment dans les rivières qui mène à l'envasement des réservoirs.

INTRODUCTION

The siltation of reservoirs is a complex and troublesome phenomenon. It presents a number of problems such as rising of bed, increasing flood levels, overflow along the banks, sediment entry into the intake and depletion of the storage capacity. It has been realized that seismic disturbances, uncontrolled deforestation, forest fire, overgrazing, improper method of tillage in the catchment area along with steep slopes of the rivers are mainly responsible for excessive sediment concentration of the inflowing water. The sediment carried by the water settles down in the reservoir and depletes its capacity. In order to ascertain the useful life of the reservoir and evaluate the economic feasibility of the projects on sediment laden streams it is essential to conduct systematic sedimentation studies of the existing reservoirs. Detailed studies have been conducted at Ichari reservoir located in Ganga Yamuna valley. The damages at Maneri resulting from silting of reservoir have also been reported.

VII.4
In the Ganga valley Maneri dam and Garhwal Chilla Hydel Scheme have been completed whereas in Yamuna valley Ichari dam and Yamuna Hydro-Electric Scheme Stage-I are under operation for the last 6 years or more. Besides a number of multi-purpose water resources projects (Fig.1) are either under planning or construction on perennial rivers and tributaries in Ganga Yamuna valley of the central Himalayas in U.P. The hydroelectric potential of the projects under construction, active consideration or investigation at present is envisaged at about 8500 MW and irrigation benefits to 2.1 million hectares. In the projects the drop will vary from 30 m to 480 m whereas installed capacity will vary from 40 MW to 2000 MW. The rivers and tributaries flow in steep slopes; the steepest tributary falls from El. 3690 to El. 1190 in a distance of about 10 km resulting in a slope of 1 in 4. The valley has been subjected to occasional massive landslides thereby blocking the river flows and creating natural lakes. These obstructions when give way in quick succession result in high uncontrolled flood waves.

THE VALLEY

Ganga Valley

The Ganga, one of the largest and sacred rivers of India, originates from Himalayan thrust where a number of rivers and tributaries taking off from Gangotri complex of glaciers join to supply continuous flow of water in the river. The two main rivers of this complex are Bhagirathi and Alaknanda which merge at Deoprayag where from it is christened as Ganga. The main tributaries of river Alaknanda are Dhauli ganga, Patalganga, Rishiganga and that of Bhagirathi Khurmola gad, Kanadia gad, Din gad and Lod gad. The catchment area of the rivers and tributaries comprises of Himalayan slopes which are in general unstable and steep with rocks highly fractured, faulted and folded. The rocks consist of a variety of geological formations ranging in age from Lower Paleozoic to tertiary. The older rocks consist of quartzites, Phyllites and slates. Lime stone forms a conspicuous lithological unit of the area exposed at some places.

Most of the tributaries of Alakanandha have their source around the Kunwarikhal ridge which is reported to have been made up of highly weathered dolomites and magnesites subject to severe temperature variation both diurnal and seasonal. Temperatures of the order of 35°C in the sun and 2°C at night can be experienced in the valley. Such large and sudden variation of temperature break up the rocks which are further disintegrated by freezing and thawing. The valley is also reported to have been subjected to frequent seismic disturbances resulting in loosening
and breaking up the rock formation thereby causing frequent landslides.

Yamuna Valley

River Yamuna originates from Himalayan thrust where a number of rivers and tributaries taking off from Yamunotri complex of glaciers join to supply continuous flow of water in the river. The main tributaries are Tons, Pabar, Girij and Ahsan. The rock formation in the region comprises of phyllites, slates, quartzites and lime stones belonging to Mandhali, Chandpur and Naghat stages. The rocks of Jaunsar series are intruded by a number of minor basic rock bodies collectively called 'Jaunsar Traps'. The rocks range in composition from Dolerite to Hornblende rhyolite. The trap rocks are generally coarse grained and highly jointed at some places; assimilation of the country rock by the intrusive body is evident.

At some places outcrops of slates and quartzites are enclosed by the traps and appear to be caught up masses of country rock generally termed as 'xenolith'. The quartzites of the xenolith are medium to coarse grained and are massive but have profuse jointing with iron oxide stains. In the river bed overburden varying from 10 m to 20 m is met with. There are many fault zones in the river bed.

ICHARI RESERVOIR

A 60 m high concrete gravity dam was constructed across river Tons, a tributary of river Yamuna, for diverting its water to Chibro underground power house through a 7 m diameter and 6.3 km long tunnel. The dam provides a small pondage utilizable to the extent of 4.7 M. cu.m (Million cubic metres) so that the discharge may be regulated according to the diurnal variation of load demand. As the river flows in a narrow gorge at the dam site the overflow section has been provided in the entire length. The spillway having its crest at El.628.8 comprises of 7 bays of 9.5 m each. It has been designed for a flood of 14800 cu.m/s. The discharge intensity at the spillway is 222.5 cu.m/s per meter which is incidentally the highest in the world. A slotted roller bucket has been provided below the spillway for energy dissipation. For the power intake a goose neck entry followed by a closed settling chamber having automatic flushing conduits was constructed. The above arrangement is a novel feature of the project.

Ichari dam was completed in 1972. Before the construction of the dam the river bed at dam site was at El.808.0. It was impounded for the first time in 1975 upto full reservoir El.644.75. The river bed conditions observed during 1976 indicated that the reservoir bed already silted upto the crest of spillway. Subsequent surveys of the year 1977, 78 and 79 show that the upstream river bed is continuously rising. Longitudinal section of the river in a reach of about 10 km upstream of Ichari dam as observed
in different years has been shown in Fig. 2. It may be seen that the sediment has deposited not only in the dead storage space, but has also encroached on the live storage. However, the river bed in a length of 0.5 km upstream of spillway is more or less the same. This is due to occasional flushing of the sediment by lifting the spillway gates. The volume of sediment deposit during the year 1976-77, 1977-78 and 1978-79 works out to 0.51, 0.12 and 1.36 M.cu.m respectively.

The observations of suspended load in the reservoir and at the spillway were also taken daily. The quantity of sediment which rolls along the bed of the reservoir near the dam could not be measured directly. However, the difference of the concentration of sediment in suspension over the spillway and that in the reservoir was taken as the rolling load. The total quantity of sediment brought by the river was calculated by adding all the figures. It was found that 1.80, 3.71 and 29.02 M.cu.m sediment was transported by 5035, 6456 and 7992 M.cu.m of the inflow of water during 1976-77, 1977-78 and 1978-79 respectively. It is worth-while to mention that during the period from 14.00 h of 2.9.78 to 24.00 h of 3.9.78 about 21.54 M.cu.m sediment was flushed over the spillway when the average spillage was of the order of 4400 cu.m/s. The average sediment concentration works out to 80,000 ppm during the above operation.

Due to excessive sediment brought by the river at Ichari dam site there is a continuous reduction in the storage capacity of the reservoir. The silting has not only affected the dead storage but has also encroached upon the live storage of the reservoir. Due to overflow of heavy sediment, abrasion damages have been observed at Ichari dam. There is a localised little abrasion on the spillway face which has been provided with a thin layer of neoprene paint. However, heavy action on the teeth is well marked. The abrasive action was so severe that the exposed reinforcement can be seen at all the sides of the teeth. The minimum damages are on the sides of the teeth while maximum action was noticed at top. However, there is no damage to the invert of the bucket and the bottom of the slots. The damages at the dents are due to the impact of shingle which rolled over the spillway after silting of the reservoir upto the crest. It has been reported that shingle of size 15 cm to 20 cm were recovered from the river bed downstream of the bucket.

Samples collected at the draft tube of Chibro power house indicate that there is no appreciable effect of silting on water conductor system. The efficiency of settling chamber for particles greater than 0.06 mm varied from
40 percent to 100 percent during the period of study.

MANERI RESERVOIR

A 39 m high concrete dam has been constructed on river Bhagirathi for diverting its flow to Uttarkashi power house through a tunnel. The spillway has 4 bays of 13 m width each separated by 4 m thick piers. A slotted roller bucket has been provided for energy dissipation below the spillway. The spillway has been designed for a discharge of 5000 cu.m/s.

River Bhagirathi witnessed an unprecedented flash flood of about 4600 cu.m/s due to the landslides in the upper valley on 6th August 1978. A discharge of about 4000 cu.m/s passed over the spillway and the balance through diversion tunnel. The movement of enormous quantity of heavy boulders and rolling debris over the spillway resulting from the siltation of reservoir caused abrasive damages on its face to varying extents.

The spillway of Maneri dam was further damaged in the floods of 1979 and 1980. As the river was flowing through bay no. 1 the extent of damage could not be ascertained. However, flow pattern indicated considerable damage there. Reinforcement bars were bared at the spillway face in bay no. 2. A cunnette of 50 cm width and 30 cm depth formed along the right side pier of the bay. The concrete around the sill beams of stoplogs and radial gates was eroded and the beams were damaged. In bay no. 3 and 4 spillway concrete was eroded to a depth of 5 cm to 15 cm.

CONCLUSIONS

Due to steep slopes and highly fractured, faulted and folded rocks the rivers and tributaries of Ganga Yamuna valley transport heavy quantity of sediment during monsoons. A total quantity of 29.02 M.cu.m of sediment was carried by the inflow of 7992 M.cu.m of water at Ichari dam site during the year 1978-79. Siltation of Ichari and Maneri reservoirs up to the crest of spillway within a short period of two years forewarns the planners of reservoir in Ganga Yamuna valley for adopting anti-erosion measures of the catchment area well in advance.

Sedimentation studies carried out at Ichari reservoir indicate that the performance of sediment exclusion device consisting of goose-neck entry followed by a closed settling tank having automatic flushing arrangement is satisfactory. The arrangement may, therefore, be provided at the power intake of rivers where large quantity of sediment are likely to be carried by the stream.
FIG. 1 - LAYOUT PLAN OF GANGA YAMUNA VALLEY DEVELOPMENT.
SEDIMENTATION IN RESERVOIRS WITH SPECIAL REFERENCE TO MATATILA RESERVOIR

SEDIMENTATION AUX RESERVOIRS AVEC REFERENCE SPECIALE AU RESERVOIR MATATILA

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ABSTRACT

All reservoirs formed by a dam on natural water courses are subject to some degree of sedimentation. The dam upsets the natural equilibrium of the stream by changing the characteristics of discharge and sediment transport capability. In order to understand the role of various parameters affecting the rate of sedimentation and pattern of deposition, sedimentation studies on some of major reservoirs in India were planned. It was under this programme that sedimentation studies at Matatila Reservoir in Uttar Pradesh were started in 1959-60 and continued up to 1974. The present paper describes the general problems of reservoir sedimentation and the various facts brought out by the actual sedimentation surveys of Matatila Reservoir.

ABSTRAIT


1. INTRODUCTION

1.1 Sedimentation in reservoirs is a complex phenomenon and cannot be predicted accurately on account of the fact that the behaviour of the various parameters affecting the process is very uncertain.

1.2 Rivers carry sediment depending upon the catchment characteristics; pattern, intensity, and duration of rainfall; flood flow; and the sediment transporting capacity of the stream. As soon as the river flow meets the reservoir created by the construction of a barrier across the stream, the transporting capacity of the flow is reduced and the sediment starts getting deposited.

1.3 Now, before coming to the actual problem of sedimentation it will be worthwhile to know as to what are the sources of sediment and how this sediment gets dislodged, because this will help us in undoing those things which are responsible for excessive sedimentation of reservoirs.
2. SEDIMENT SOURCES

2.1. The sediment problem arises from man's disturbance of the prehistoric geological pattern. Cutting and burning of bushes on lands and forests, over grazing of grass lands, and cultivation of crop-lands have greatly reduced the primeval vegetal production. The erosion of the soil mostly by impact of rain drops, wind, and surface runoff has been much more than the soil formation.


3. SEDIMENT TRANSPORT IN INDIAN RIVERS

3.1 The rivers of India can be divided into two main categories on the basis of their sediment yield, namely,

(i) Rivers of the Himalayan Region.

(ii) Rivers of the Non-Himalayan Region.

3.2 Rivers of Himalayan region contribute as much as 70% of the total water and also carry far more sediment and much coarser material than the Non-Himalayan rivers. Reasons for higher rate of sedimentation in Himalayan rivers are:

(i) Unstable and steep slopes.

(ii) Highly fractured, faulted, and folded rocks.

(iii) High seismicity along mountain ranges.

(iv) Heavy rainfall.

(v) Severe temperature variation resulting in breaking of rocks due to freezing and thawing.

(vi) Development of rapid communication network resulting in numerous and heavy landslides.

(vii) Conducive situation for snow avalanches.

3.3 Due to above reasons, sometimes, the rivers in these reaches get dammed up and these dams after some time give way releasing large amounts of water and sediment down below. In one of such valleys, known as Alakmanda valley, it is believed that there is a slide zone approximately every ten kilometres apart. A major slide occurred in this valley in the year 1893 creating a 275 metre high dam across Biraahi Ganga. Another major slide in the valley occurred in 1970 creating an artificial dam which on breaching caused heavy devastation in the valley and heavy siltation in more than a century old canal. The silt clearance needed for recommissioning of the canal was about 1.7 million cubic metre costing about Rs.10.00 million.

4. THE PROBLEM OF RESERVOIR SEDIMENTATION

4.1 All reservoirs formed by a dam on natural water courses are subject to some degree of sedimentation. The planners are confronted with the problem of prediction of the rate of sedimentation and the probable time when the reservoir would be affected in discharging its useful functions. The main problem created by silting of reservoirs is the loss of storage associated with loss of useful life, effect on outlet sill elevations, and recreational facilities etc.

4.2 The total sediment which is likely to be deposited in a reservoir depends mainly on the following two factors:

(i) The ratio of capacity to inflow.

(ii) Catchment characteristics of the reservoir.

The modifiers to the above factors are:

(i) Trap efficiency of the reservoir.

(ii) Shape of reservoir.

(iii) Character of sediment.

(iv) Method of reservoir operation.

5. ESTIMATION OF SEDIMENT DEPOSITS.

5.1 The estimation of expected sediment deposits in a reservoir can be done by one or more of the following methods:

(i) Field inspection of watershed and comparing it with other areas.

(ii) Stream flow sampling.

(iii) By applying results of resurveys of existing reservoirs.

6. MATATILA RESERVOIR AND ITS SEDIMENTATION

6.1 Matatila Reservoir.

The Matatila reservoir was created by constructing an earth dam across river Betwa at Matatila in District Jhansi of Uttar Pradesh (India). It was designed for a storage capacity of 1132.7x10^6 m³ at the full reservoir level of 308.46 m. It was expected that an effective storage of 768.5x10^6 m³ would be available after 100 years excluding yearly evaporation.
losses of \(85 \times 10^6\) m\(^3\) and the total silting loss of 283.2 \times 10^6\) m\(^3\). It provides facilities for irrigation, power generation, water supply and fish cultivation.

The dam was completed in 1956 with the spillway up to the crest level at EL. 301.45 m. The spillway gates were erected later and the reservoir could be filled up to full reservoir level for the first time in 1964. The reservoir has been designed to have a storage capacity of about one sixth of the average annual runoff and gets filled up even in the leanest year.

6.2 The Geology of the Catchment

The river flows through the plateau composed of the Bundelkhand granite which occasionally forms low flat dome shaped hillocks rising to small heights not more than a few hundred metres from the surrounding area. The general features of the ground are gently undulating sparsely cultivated uplands with shallow valleys and alluvial plains. The soil is of low fertility covered in parts with scanty jungle and low trees.

The Bundelkhand granite is medium to coarse grained typical granite. Much of it is homogeneous for considerable thickness and extent. On weathering it does not tend to form boulders, a usual feature of the granite, but gives smooth bare surfaces with a thin weathered crust.

At the dam site, the river cuts through the granite country and flows through a wide and shallow valley. Moorum (laterite) and clayey soils form a covering of varying thickness over the rock on either bank.

6.3 Sedimentation Studies

The loss in capacity of Matatila Reservoir has been determined by two methods:

(i) Inflow-outflow method.

(ii) Hydrographic survey.

Inflow-Outflow Method

The loss in capacity by this method has been found by calculating the silt content of the water at two points, viz.,

(i) When it enters the reservoir.

(ii) When it leaves the reservoir.

This involved river gauging for discharge and sediment measurements. At high floods, the discharge and sediment measuring equipment could not be lowered to desired depths, but instead, started floating on surface. Under such circumstances, the discharge was calculated from the extrapolated stage discharge curve, and sediment samples could be collected from the surface only. This made the results obtained by this method less accurate.

Undisturbed sediment samples from the reservoir bed were also collected and analysed with a view to establish volume-weight relationship for sediment load entrapped in the reservoir so that the entrapped sediment estimated from inflow-outflow observations might be converted into volume to find loss in capacity. The analysis of these samples indicated variation in sediment size and density along the reservoir.

Hydrographic Survey

It is the direct method for determining the depth of sediment deposit, the pattern of sediment distribution, and capacity loss of the reservoir. The depth gauging under water is done by echosounding equipment. The bed configuration is recorded automatically on a calibrated chart. The hydrographic surveys of Matatila reservoir have been conducted six times so far in the years 1962, 1964, 1966, 1969, 1971 & 1974.

6.4 Result of Sedimentation Studies

By inflow-outflow method

The analysis indicated that the inflow of the sediment generally increases with the increase in runoff. It is also indicated that sediment concentration is higher in the first flood peaks. The runoff, sediment concentration and the quantity of sediment brought with the flood peaks is given in Table 1. It indicates that major portion of sediment is generally brought with flood peaks only.

The trap efficiency was calculated every year and it has been found varying from 70 to 90 percent. The trap efficiency in 1971 & 1972 has been found to be low, as the first few flood peaks in these years were passed through the spillway without absorption. This indicated that the time and stage of flood at which the reservoir is filled has a marked influence on the trap efficiency and consequently on loss in capacity of the reservoir.

The grain size distribution analysis of the samples reveal a small percentage of sand indicating that only a small portion of the sediment moves as bed load.
<table>
<thead>
<tr>
<th>Year</th>
<th>Period</th>
<th>Maximum sediment concentration gm/litre</th>
<th>Sediment Inflow Million tonnes'</th>
<th>Percentage of the total</th>
<th>Water Inflow Million cu.m.</th>
<th>Percentage of total</th>
</tr>
</thead>
<tbody>
<tr>
<td>1965</td>
<td>July 27, 1965 to August 5, 1965.</td>
<td>2.96</td>
<td>6.86</td>
<td>61.2</td>
<td>3,415</td>
<td>33.6</td>
</tr>
<tr>
<td></td>
<td>September 3, 1965 to September 10, 1965.</td>
<td></td>
<td></td>
<td></td>
<td>1,178</td>
<td>22.5</td>
</tr>
<tr>
<td>1966</td>
<td>July 29, 1966 to August 5, 1966.</td>
<td>5.98</td>
<td>4.33</td>
<td>69.0</td>
<td>960</td>
<td>33.6</td>
</tr>
<tr>
<td></td>
<td>August 17, 1966 to August 23, 1966.</td>
<td></td>
<td></td>
<td></td>
<td>1,242</td>
<td>43.7</td>
</tr>
<tr>
<td>1968</td>
<td>July 31, 1968 to August 9, 1968.</td>
<td>2.90</td>
<td>2.84</td>
<td>44.2</td>
<td>1,675</td>
<td>36.7</td>
</tr>
<tr>
<td></td>
<td>August 15, 1968 to August 21, 1968.</td>
<td></td>
<td></td>
<td></td>
<td>1,515</td>
<td>33.0</td>
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<tr>
<td>1970</td>
<td>August 30, 1970 to September 1, 1970.</td>
<td>2.29</td>
<td>4.26</td>
<td>35.9</td>
<td>2,888</td>
<td>30.2</td>
</tr>
<tr>
<td></td>
<td>September 3, 1970 to September 15, 1970.</td>
<td></td>
<td></td>
<td></td>
<td>2,390</td>
<td>25.0</td>
</tr>
<tr>
<td>1971</td>
<td>July 19, 1971 to August 2, 1971.</td>
<td>2.26</td>
<td>11.07</td>
<td>63.5</td>
<td>7,566</td>
<td>58.6</td>
</tr>
<tr>
<td></td>
<td>September 6, 1971 to Sept. 11, 1971.</td>
<td></td>
<td></td>
<td></td>
<td>2,062</td>
<td>16.0</td>
</tr>
<tr>
<td>1972</td>
<td>August 16, 1972 to August 21, 1972.</td>
<td>4.95</td>
<td>2.70</td>
<td>40.1</td>
<td>2,143</td>
<td>37.8</td>
</tr>
<tr>
<td></td>
<td>August 27, 1972 to September 4, 1972.</td>
<td>1.40</td>
<td>2.64</td>
<td>39.2</td>
<td>2,346</td>
<td>41.3</td>
</tr>
</tbody>
</table>

By Hydrographic Survey

The longitudinal bed profiles obtained at the end of various hydrographic surveys have been superimposed on the original bed profile of the reservoir and are shown in Fig. 1.

The results of the hydrographic survey indicate that by the end of 1974, the reservoir had lost 10.52% of its capacity, the average annual loss being 0.80%. The entire sediment has not gone into dead storage only as envisaged at the time of project planning. Instead, 43% has found its way into dead storage and remaining into live storage. The average sedimentation index works out to three times the value adopted at the time of project planning.

The results of these surveys have been used to estimate capacity of the reservoir in coming years by using Empirical Area Reduction method. The variation of dead storage, live storage, and total capacity of the reservoir with time are shown in Fig. 2. It is seen from these curves that the reservoirs is expected to be filled up to the dead storage level and the crest level by the year 1996 and 2026 respectively. However, this trend is likely to be appreciably modified after the construction of proposed Rajghat Dam upstream of Matatila Reservoir on the same river.
7. SEDIMENTATION SURVEYS OF OTHER RESERVOIRS AND THEIR FINDINGS

7.1 Systematic sedimentation surveys of reservoirs were started in India in 1959 and since then they are in progress. Some of the important findings of these surveys are as below:

(i) The annual rate of siltation from a unit catchment has generally been more than assumed at the time of project planning and has been found in some cases to be as high as nine times.

(ii) The sediment has deposited, not only in dead storage, but has also encroached on live storage. Due to this, certain important aspects of design such as dead storage level, outlet sill level and openings of penstocks etc. are likely to be modified.

(iii) In basins where full integrated development has yet not been completed, the sedimentation rate in the downstream reservoirs have been higher consequent on not having fully implemented the entire scheme of building up of upstream dams. This high rate of silting in the downstream reservoirs emphasizes that in an integrated plan, the construction of upstream reservoirs should not be unduly deferred.

(iv) Because the actual silt load coming into the reservoir is far more than estimated, there is a great need for adoption of better samplers and techniques for sediment sampling. The bed load which sometimes forms an appreciable part of the total load, needs gauging as well.

(v) The density of the sediment coming into the reservoir has been found to vary between wide limits and as such there is need for correct information on density of deposited sediment.

(vi) Small reservoirs which are meant to cater for diurnal variations are likely to be silted up till the crest level within a couple of years.

(vii) The silting of the reservoirs can be reduced by proper planning of reservoir operation. It may be done so as to:

(a) allow maximum consolidation of deposited sediment.

(b) allow the first few flood peaks to pass down the spillway without absorption. The first few floods contain the maximum amount of sediment.
(c) Flush the silt deposited in the live storage into dead storage.

(viii) The data also points out that due thought is necessary towards capacity surveys of reservoirs right from the time of project planning itself. The job should be planned well in advance and the basic surveys including layout of ranges, fixing range monuments, levelling of ranges etc. should be completed before impounding of dam. As a matter of fact, the hydrographic surveys should be started right from the stage of coffer dam construction, because, it would help in estimating bed load and silt load separately. Also, this would help to adopt measures for reducing sedimentation in advance so that these measures are effective when the reservoir starts impounding.

(ix) Sediment surveys in reservoirs with different characteristics could provide useful data to develop design curves for determining sediment distribution pattern on regional basis.

8. MEASURES TO REDUCE SEDIMENTATION IN RESERVOIRS

8.1 A good vegetative cover on a watershed is the best preventative of sedimentation. Soil conservation measures such as:

(i) Crop rotation
(ii) Strip cropping
(iii) Control of depletion of natural vegetation by
   (a) regulating grazing and fire control.
   (b) reforestation
   (c) seeding of pastures
   (d) Planting the perennials on eroded and gullied areas
   (iv) terracing of land providing terrace outlets
   (v) check dams etc.

can be used to reduce sediment flow in stream. Though in some basins these soil conservation measures have been used for sediment control, the quantitative data is very little to show the effect of these
measures in controlling silt.

It is suggested that soil conservation measures in those areas where reservoirs are proposed should immediately be started at least in few representative watersheds and the effect of these measures be regularly monitored. This would help in planning of such measures at a larger scale.

8.2 Removal of Sediment deposits in reservoirs by excavation, dredging, draining and flushing either by mechanical or hydraulic methods. These methods are very difficult and less practical.

8.3 By employing special methods such as locating the storage basins off channel and filling it by diversion.

8.4 By suitably adjusting the reservoir operation such that water is drawn off when sediment is still in suspension.

8.5 Since the project involves a very heavy stake, every single year which is added to the useful life of the reservoir is worth the cost and trouble involved in its achievement.

REFERENCES


ASSESSMENT AND CONTROL OF FACTORS CONTRIBUTING TO SILTMATION OF THE UKAI RESERVOIR, INDIA

EVALUATION ET CONTROLE DES FACTEURS QUI CONTRIBUENT A LA SEDIMENTATION DU RESERVOIR D’UKAI, INDE

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ABSTRACT

A 68.58 m high dam on the Tapi river with a gross storage capacity of $851 \times 10^3$ ha m and water spread of 520 sq km was completed in 1971 for flood control, irrigation and power. The computed silt index in 1971 was 1.37 ha m/100 sq km/year but the recent studies have indicated higher rate of siltation.

The rocks exposed in the catchment area are mostly the Deccan basalt with extensive residual and alluvial soils. The paper highlights factors that control sedimentation in the Ukai reservoir for planning suitable remedial measures.

ABSTRAIT

Le barrage de 68.58 m de haut sur la rivière Tapi, avec une capacité de réserve de $851 \times 10^3$ ha m et une surface de 520 sq km a été terminé en 1971 aux fins d’irrigation, d’électricité et de contrôle des inondations.

L’index d’envasement calculé en 1971 était de 1.37 ha m/100 sq km/par an mais des études récentes ont montré un envasement plus important.

Le rocs exposé dans la surface de captation des eaux est surtout le basalt év Deccan avec des éboulis de sédiments résiduels et d’alluvions. Cet article met en lumière les facteurs qui contribuent à la sédimentation dans le réservoir d’Ukai afin qu’on puisse prendre des mesures de correction.

INTRODUCTION

A 68.58 m high and 4.9 km long composite dam on the Tapi river has been constructed for flood control, irrigation and generation of hydropower. The dam commissioned in 1971 forms a reservoir, which is spread over an area of 520 sq km with a gross storage capacity of $851 \times 10^3$ ha m. The computed silt index in 1971 was 1.37 ha m/100 sq km/year, but preliminary sedimentation observations carried out recently have indicated 31 ha m/100 sq km/year indicating a loss of about $194 \times 10^2$ ha m in reservoir capacity over a period of ten years. This high rate of siltation in the Ukai reservoir has caused concern to the engineers and geologists and a survey of the catchment area was carried out to study the features which are responsible for silt production and transportation (Bansode et al. 1980, 1981). In this paper an attempt has been made to discuss the various parameters that contribute to the siltation of the Ukai reservoir for planning suitable remedial measures.

Physiographic features

The Tapi river, 720 km long, is one
Fig. 1. Drainage map of the Ukai catchment area:

1 - Isohyetal line (mm); 2 - Spot height in m; 3 - Subcatchment.

I Upper Tapi sub-catchment     II Purna sub-catchment

III Panjhara - Vaghur sub-catchment     IV Northern sub-catchment
of the two major rivers of Central India, that flows from east to west. The river occupies a rift valley between the Mahara-
shstra plateau and Sahyadri ranges on the south and the Satpura ranges on the north. The river has its origin near Multai at an
elevation of 823 m above sea level and flows for about 186 km through the Satpura ranges upto Burhanpur after which the valley
widens. The bed levels of the river between Burhanpur and Shirpur range from 150-300 m. Beyond Shirpur the levels fall
below 150 m and near the dam the river bed is at an elevation of 60 m. The river debouches into the Arabian sea 125 km west
of the dam.

The catchment area of the Ukai reservoir is 62,224 sq km. One-third of the catchment area lies on the north, whereas
the remaining two-thirds of the catchment is on the south of the Tapi. Purna river, the most important tributary of the
Tapi joins it near Shriyamal. Other major tributaries on the south side are the Girna, Panjhra, Waghur and Bordy. The
Girna and Anker join the Tapi river from the north. In addition, there are hundreds of streams debouching into the river before it
enters the terminal (Fig. 1) gorge near Ukai, where the dam has been constructed.

There are three physiographic units viz. the dissected Deccan plateau varying in heights from 600 to 1500 m above the sea
level and forming the water divides between the Narmada on the north and Godavari on the south. The intervening undulatory
terrain lies at heights of 300-600 m and has moderate slopes with widths varying from 2.5 to 10 km. Flat gently sloping
extensive flood plains between elevations of 60 and 300 m above the sea level has maximum 10 km width in the Tapi valley,
whereas the maximum width of the flood plain in the Purna valley is 60 km. The combined length of the Tapi and Purna
flood plains, upstream of the Ukai dam is about 360 km.

Rainfall and discharge

The annual rainfall in the catchment area is 775 mm, the maximum being around 2000 mm in the Satpura ranges in the
Upper Tapi valley. The rainfall is restricted to the monsoon period between June and September, when the discharge in the
river swells to around 11,000 cu m/sec. The maximum observed flood discharge in 1968 was 42,450 cu m/sec. During the post-
monsoon period, the flow in the river is less than 300 cu m/sec, dwindling down to 10 cu m/sec in May.

Soils and land use

Most of the catchment area is covered with black soils with variable thickness. The Archeans and Gondwanas are exposed in
a small area in the northeast part of the basin and form red and black soils. On the higher reaches there is a thin cover of
black soil. But, on the slopes black soils with one to three meters is met with. In the Purna and Tapi flood plains there is a
thick accumulation of black soil exceeding 30 m in depth (Fig. 2). The Tapi flows at a considerable depth below the alluvial
plains. At places, large patches of ancient alluvium have been noticed above 150 m elevations. In alluvial deposits of the
Purna valley, for a distance of 50 km between Akola and Arravati, brine has been met with in the dug wells at depths of
about 40 m (Pascoe, 1973).

The upper areas of the Satpura, Gawai-
garh and Shyadri ranges are generally covered with forests whereas the inter-
mediate zone and the flood plains provide fertile soils, where extensive cultivation is practiced (Fig. 3). A small area in
the southwest part of the basin is covered with grass and scrub. The catchment area is thickly populated with a number of import-
ant towns located in it.

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Fig. 2. Soil map of Ukai catchment area:
1 - Deep black soil - Pellusterts, Chromusterts, Pelluderts;
2 - Medium black soil - Pellusterts, Chromusterts;
3 - Shallow black soil - Ustortherts, Calcorthids;
4 - Mixed red and black soils - Association of Affisols and Vertisols.

<table>
<thead>
<tr>
<th>Land-use</th>
<th>Percentage of the catchment area</th>
<th>Susceptibility to erosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grass &amp; Scrub</td>
<td>3%</td>
<td>Low</td>
</tr>
<tr>
<td>Forest</td>
<td>41%</td>
<td>Moderate</td>
</tr>
<tr>
<td>Arable</td>
<td>56%</td>
<td>High</td>
</tr>
</tbody>
</table>

Table. Land use in the catchment area of Ukai reservoir (After Irrigation Atlas of India 1972).
A number of hot springs and earthquakes have been located in the catchment area. Geomorphic studies carried out in lower reaches of the Tapi have indicated evidences of sub-Recent movements in the area (Bedi, 1974). These features show that the Tapi flows in a tectonically active belt.

Siltation of Ukai reservoir

Siltation is an important factor which determines the long-term competency of a reservoir. Siltation is due to soil erosion which depends upon the lithology, weathering of rocks, stability of slopes, stream gradients, precipitation and lack of vegetation in the catchment area. In order to study the siltation problem and identify areas of silt production the entire catchment has been divided into four sub-catchments which have distinct characteristics (Fig. 1).

Tapi sub-catchment upstream of Burhanpur

The sub-catchment occupies an area of about 9,000 sq km. The river flows through hilly terrain where the Archean and Gondwanan rocks are exposed in the upper reaches, but in most of the area, Deccan basalt forms the bed rock and are weathered to variable degree forming shallow to medium black soil. The tributary streams have high gradients and the area receives the highest precipitation. The slopes are covered with thick forest.

Purna sub-Catchment

The Purna sub-Catchment between Gavilgarh ranges on the north and the Ajanta ranges on the South, occupies an area of 20,000 sq km. The Purna has formed about 50 km wide and 160 km long flood plains. The general slope of the flood plain is 52 m in 125 km. The valley slopes are gentle. The streams have dendritic

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**Fig. 3.** Land use map of Ukai catchment area:

1 - Grassland and scrub; 2 - Forest; 3 - Arable.

Geology of the catchment area

In the upper reaches, the Tapi cuts through a small patch of Archeans and the Gondwanan rocks, but rest of the catchment area, in the Satpura range and the Maharashtra Plateau is covered with Deccan basalt flows with extensive alluvial plains (Fig. 4). The basalt flows are generally horizontal in their disposition but are intersected by innumerable joints, fractures and dykes of dolerite and gabbro due to which the hills are linear in extent and do not show the characteristic flat landform of Deccan plateau. The fractures, joints and the dykes are aligned in ENE-WSW direction parallel to the Narmada-Son deep seated lineament. In addition lineaments trending in NNE-SSW AND NW-SE direction representing the west coast and the Godavari tectonic trends are also seen in the area (Mehta, 1961). The Tapi river flows, along a series of enechelon faults and fractures in ENE-WSW direction and marks southern boundary of the well known Narmada-Tapi rift zone. Due to down faulting along these lineaments the Tapi and Purna have formed wide valleys with gentle slopes on either side and flat flood plains filled in with unusually thick alluvium.

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**Fig. 4.** Geological map of the Ukai catchment area:

1 - Alluvium, soil; 2 - Deccan Basalt; 3 - Gondwanas; 4 - Archeans.
drainage pattern in the upper slopes and sub-parallel drainage pattern in the lower reaches. More than 90 percent of the area is covered with thick black soil, which is extensively cultivated. Due to high rainfall in the area is subject to sheet and gully erosion. A number of dams have been constructed on the tributaries from the south, but these are located in the upper reaches.

Girna, Panjhora, Bordy and Vaghur sub-Catchment

These are separate sub-catchments but have been grouped together as they have similar characteristics. The combined catchment area is about 21,750 sq km. The streams originate from the Sahyadri range and flow from west to east and then swing towards north to join the Tapi. The streams are perhaps controlled by lineaments parallel to Narmada-Tapi and West Coast rifts. The area is covered with deep black to medium black soils which are under cultivation. The Girna, Panjhora and Bordy appear to have large alluvial fans which have coalesced to form extensive plains on the southern side of the catchment. However, these fans cannot be recognised due to extensive agricultural activity. The irrigation is carried out through canals and dug wells. Like Purna sub-catchment, the Girna - Vaghur sub-catchment is subject to sheet and gully erosion and is a potential source of silt which is transported into the Ukai reservoir during high floods. Some amount of silt is, however, trapped in the dams constructed across the Girna and other streams in the upper reaches.

Northern sub-Catchment

The entire catchment area on the northern flank downstream of Barhanpur forms a linear belt known as the 'Khandesh' plains. Aner and Gomai are the important streams of the northern catchment area. Most of the streams rise from the southern slopes of E-W Satpura range. 1000 - 1300 m above the sea level and join the Tapi around 300 m elevation. The drainage pattern is dendritic to sub-parallel and is generally controlled by the slope of the area. The Deccan basalt are weathered and yield medium black soil which form the plains. The Tapi plains have steep slope of 42 m in 48 km i.e. nearly 1:1000 up to Bhusaval. This linear catchment is 50 km in width and about 200 km in length. The upper catchment is forested whereas the plains are arable. Due to steep gradient of streams and moderate to heavy rainfall, the area is prone to channel and gully erosion.

Silt Analysis

Mechanical analysis of the silt samples indicate the deposition of mostly medium to fine sediments all around the periphery of the reservoir between levels 86 and 105 m above sea level. Average density of the sediments in the reservoir is 1460 kg/cm³.

The rate of siltation in storage reservoirs depends upon the capacity inflow ratio and content of the inflow. The capacity of the Ukai reservoir is 851 x 10³ ha m and the annual run off is 1.722 x 10⁶ ha m which gives capacity inflow ratio 0.49. According to Brune’s Curve, the Ukai reservoir has about 95 per cent trap efficiency (Brune, 1953).

CONCLUSIONS

A large part of the catchment area of the Ukai dam is covered with medium to deep black soil. Extensive stretches of deep black soil in the Purna and Tapi valleys lie at elevations varying from 100 - 500 m. The area receives moderate to high rainfall varying from 750 to 1600 mm in a year. The main streams originate at an elevation around 800 - 1200 m above the level and join the Tapi river around 100 - 300 m elevation and thus have moderate to low gradients with good silt carrying capacity. The Tapi and Purna occupy a wide rift valley due to down faulting along a series of enechelon faults. The river has formed extensive flood plains. The valley slopes covered with residual soil and the plains having thick alluvial soil are subject to gully and sheet erosion.

As the catchment area of the Ukai reservoir is very large, the silt produced in the farther sub-catchments would get deposited erratically. Thus the silt produced in sub-catchments nearer to the reservoir is transported and deposited in it during flood seasons. The Ukai reservoir has high trap efficiency. It is observed that even during the flood seasons only a small amount of silt passes over the Ukai dam.

There are number of dams across the streams of sub-catchment on the southern side but these are located at higher reaches and may not be acting an effective silt trap. Taking into consideration the type of the Tapi and Purna catchments, efficient land management and afforestation seem to be the most suitable measures to control sheet erosion and prevent excessive siltation of the Ukai reservoir. Suitable sites for construction of check dams may also be investigated.
ACKNOWLEDGEMENTS

The authors are grateful to the Gujarat Engineering Research Institute, Vadodara for the silt data of the Ukai reservoir.

REFERENCES


L’ENVASEMENT DE LA RETEUNE DU CHAMBN (ALPES FRANCAISES) APRES UN DEMI-SIECLE D’EXPROTATION

THE SEDIMENTATION IN THE RESERVOIR OF THE CHAMON’S DAM (FRENCH ALPS) AFTER HALF A CENTURY OF EXPLOITATION

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ABSTRACT

Le barrage du Chambon, situé sur la Romanche à l’amont de Grenoble, a été mis en eau en 1935. Il s’agit d’un barrage-poids en béton de 90 mètres de haut. Le bassin versant de la retenue a une superficie de 336 km². Il intéresse essentiellement une zone de haute montagne à relief très accusé : escarpements rocheux granitiques, alpages sur schistes alterables et moraines.

Les dépôts argilo-silex litsés sédimentés dans la retenue, d’un volume initial de 50 millions de m³, ont atteint 18 mètres d’épaisseur au pied du barrage. Le volume des sédiments piégés dans la retenue a été évalué après la vidange totale de 1981. Les carottes obtenues à cette occasion permettent d’étudier la succession lithologique, la composition minéralogique et les caractéristiques géotechniques des sédiments déposés depuis 45 ans.

ABSTRACT

The Chambon’s dam on the river Romanche near Grenoble was completed in 1935. It is a concrete gravity-dam, 90 meters high. The watershed (336 km²) mainly consists of a high mountain zone with granitic rocks and mountain pastures on shales and glacial deposits. The lake, whose initial volume was 5.10⁷ m³ is partially filled with layers of clay and silt up to 18 meters thick behind the dam.

The total volume of materials deposited in the lake has been calculated after emptying the reservoir in 1981. The opportunity to obtain core samples was used to study the lithology, the mineralogy and the geotechnical characteristics of the sediments deposited since 45 years ago.

Cette étude a été réalisée dans le cadre d’une recherche financée par Electricité de France, Groupe de Production Hydraulique Alpes, à l’occasion d’une vidange totale du barrage du Chambon.


Les principales caractéristiques de cet aménagement sont les suivantes :

- superficie du bassin versant : 336 km² dont 254 km² pour la Romanche et 82 km² par captage du Ferrand,
- capacité utile de la retenue : 50 millions de m³,
- cote normale de la retenue : 1040 m,
- cote du lit de la Romanche au barrage : 950 m,
- vidange de fond d’origine, cote du seuil de l’entrée : 951.48 m,
- débit maximal : 90 m³/s.
Figure 1 : Fluctuation de la cote du plan d'eau : valeur moyenne (1950-1980) et vidange totale (1980).


En 1962, une nouvelle vidange de fond a été construite à la cote 959.45 avec un débit maximal porté à 110 m³/s. A l'occasion des travaux de réaménagement de la chute de Saint-Guillermé, deux vidanges complètes de la retenue ont été effectuées (1980 et 1981). La première vidange a permis d'observer, au pied du barrage une épaisseur de 18 m (cote 908) de dépôts argilo-silteux lités plus ou moins consolidés.

Du fait des vidanges totales et des travaux effectués pour modier les caractéristiques de la vidange de fond, l'histoire de la sédimentation dans le réservoir est complexe. Nous avons cependant tenté dans ce qui suit d'étudier les sédiments déposés dans la cuvette du Chambon pendant les 45 années de fonctionnement du barrage, aussi bien sous l'aspect quantitatif que qualitatif.

I. CONTEXTE GÉOLOGIQUE DU BASSIN VERSANT

1) Le bassin versant de la Romanche (fig. 2).

Du point de vue géologique, le bassin versant de la Romanche au Chambon (254 km²) est constitué de trois types de terrains :
- le massif cristallin de la Meije (3 983 m) surtout présent en rive gauche. Du fait de sa haute altitude il est recouvert en partie par des glaciers,
- les terrains sédimentaires (Lias et Flysch des Aiguilles d'Arves) à dominante schisteuse bien développés en rive droite,
- les terrains de couverture (moraines et éboulis).

La répartition surfacique de ces différents terrains est la suivante :
- terrains cristallins : 47 % dont 11 % recouverts de glaciers,
- terrains sédimentaires : 43 %,
- terrains de couverture : 10 %.

L'examen du cadre géologique et des photographies aériennes permet de faire trois remarques concernant l'érosion dans le bassin versant :
- le Cristallin, malgré son relief accentué, est peu érodable,
- l'érosion se développe surtout en rive droite, dans les formations tendres et schisteuses du Lias (37 % de la surface du bassin versant). Elle est particulièrement forte dans les zones où il n'y a pas de couvert végétal,
- les terrains de couverture sont fortement érodables.

2) Le bassin versant du Ferrand

Dans le bassin versant du Ferrand (83 km²) on retrouve des terrains analogues aux précédents, le Cristallin de la Meije étant remplacé par celui du massif des Grandes Rousses. Néanmoins, les terrains sédimentaires à dominante schisteuse y sont
nettement majoritaires. Aussi pour l'ensemble du bassin versant alimentant le réservoir du Chambon (336 km²), la proportion des terrains facilement érodables est de l'ordre de 60%.

II. CONTEXTE CLIMATIQUE ET HYDROLOGIE

1) Les facteurs climatiques de l'érosion.

On dispose de données climatiques précises pour la zone de haute montagne comprenant le bassin versant du Chambon depuis la mise en service en 1962 de la station météorologique de la Grave (altitude 1785 m).

En ce qui concerne les précipitations, la moyenne mensuelle ramenée en hauteur d'eau est de 75 mm avec deux périodes notablement plus humides, l'une en février et mars (85 mm), l'autre surtout au mois de novembre (115 mm). Il n'y a pas véritablement de période sèche, le minimum s'établissant en général en mai ou en octobre avec une pluviométrie moyenne de 65 mm. La moyenne annuelle, calculée sur une période de vingt années de fonctionnement de la station, s'établit à 920 mm.

Les variations de température sont importantes aussi bien sur le plan saisonnier que journalier. En hiver et au printemps les gelées sont fréquentes. On note une moyenne de 91 jours de gel par an et de 20 jours sans dégel. La température moyenne est de 4.3°C à l'altitude de la station.

L'alternance des cycles gel-dégel contribue largement à la désagrégation mécanique des terrains, tandis que les températures estivales (moyenne des maximums en juillet : 18°C) favorisent la dessication des sols et des matériaux argileux. Les fragments ainsi élaborés, surtout dans les terrains schisteux, sont de petite taille et peuvent être facilement transportés par les eaux de ruissellement.

Par comparaison avec d'autres bassins versants voisins, dans lesquels des mesures ont été effectuées (VIVIAN, 1979), on peut indiquer que le transport solide (exprimé en charge solide par volume d'eau) doit présenter deux pics ; le premier en juin, le second plus important en septembre-octobre. Il est intéressant de noter que ce deuxième pic n'accompagne pas les plus forts débits qui se produisent au moment de la fonte des neiges (juin-juillet). Au printemps la persistance du manteau neigeux protège le sol et l'érosion ne se fait sentir nettement.
qu'à partir du mois de juin, lorsque le sol est découvert... À l'automne, la charge solide atteint un maximum lorsque de fortes précipitations se produisent sur les sols et les terrains schisteux ayant subi une forte dessiccation. Enfin, l'érosion et le transport solide sont minimes en hiver, le seul apport momentanément important pouvant être celui d'éléments éboués dans le lit et transportés par charriage.

2) Hydrologie
On connaît, depuis plus de cinquante ans, les débits liquides transitant dans la Romanche au niveau du Chambon grâce à une station de jaugeage E.D.F. installée en amont de la retenue à la cote 1057. La moyenne des débits instantanés sur la période 1936-1980 s'établit à 7,5 m³/s pour un bassin versant de 220 km².

La station du Pont de Mizoën sur le Ferrand (altitude 1166) indique un débit moyen de 2,75 m³/s pour 79 km² de bassin versant. Les eaux du Ferrand ont été captées et ramenées dans le lac du Chambon par une galerie, en service depuis 1949. Pour l'ensemble du bassin versant de la retenue (336 km²), le débit moyen instantané, depuis cette date est de 11 m³/s.

L'étéage se situe en hiver en général au mois de février (moyenne de 1,3 m³/s pour la Romanche et de 0,7 m³/s pour le Ferrand). Les débits moyens maximum, observés à la fonte des neiges en juin-juillet, sont de 19 m³/s pour la Romanche et de 7,5 m³/s pour le Ferrand. Enfin les crues les plus importantes peuvent se produire dans les deux bassins au moment des orages de l'été (qui se superposent alors à une fonte importante) ou de ceux de l'automne. Pendant toute la durée de fonctionnement de la station du Ferrand, la crue maximale s'est ainsi produite en août 1958 après un violent orage. Le débit instantané maximum était de 70 m³/s soit 25 fois le débit moyen et 100 fois celui du mois d'étéage.

III. ESTIMATION DU VOLUME D'ALLUVIONS
La finalité de cette étude est l'évaluation d'une part du volume d'alluvions piégées dans la retenue et d'autre part du volume libéré par le turbinage et les vidanges successives depuis la mise en eau du barrage.


Figure 3 : Morphologie du fond de la retenue lors de la vidange de 1981.

Vingt huit coupes transversales à la retenue permettent de mesurer en mètres carrés, selon ces sections, la quantité d'alluvions en présence. Une deuxième planimétrie aboutit aux volumes cherchés (fig. 5). La projection Lambert et la cote N.G.F. 1040 (cote maximum du lac) sont utilisées pour une bonne superposition des coupes établies à partir des deux cartes.

L'épaisseur et la nature des alluvions ont été précisées par des mesures et essais sur place : 15 sondages à la tarière à main, 5 sondages carottés, 9 pénétromètres, 1 sondage électrique, 1 sondage sismique, 8 relevés de coupes en bordure de terrasses.
Figure 4 : Coupes montrant le taux d'envasement observé lors de la vidange de 1981.

Figure 5 : Estimation des volumes d'alluvions admis puis évacués ou conservés dans la retenue du barrage du Chambon. Pour chaque abscisse (distance à l'axe du barrage), l'ordonnée (volume en mai 1981) représente la surface figurée en grisé sur les coupes de la figure 4.

Pour obtenir une valeur voisine de l'alluvionnement depuis la première mise en eau, les terrasses ont été prolongées sur chacune des coupes, considérant que ce niveau aurait été atteint sans les turbinages et vidanges successives. Cette valeur est par défaut du fait qu'une certaine quantité d'alluvions des terrasses est entraînée lors des turbinages et vidanges.

Par cette méthode, le volume des alluvions en place actuellement dans la retenue est estimé à $3.8 \times 10^6$ m$^3$. La quantité des matériaux solides évacués du fait des turbinages et vidanges depuis la première mise
Figure 6 : Coupe détaillée du prélèvement 1 correspondant à l'ensemble du dépôt depuis 1935 (en blanc le matériau argilo-silex, en noir les niveaux plus grossiers). 
ω nat. = teneur en eau naturelle.
en eau peut être évaluée à $1.6 \times 10^6$ m$^3$. Aussi, on peut estimer que le volume total des matériaux sédimentés dans la retenue depuis la première mise en eau est d'environ $5.4 \times 10^6$ m$^3$. Ces valeurs exprimées en poids de matière sèche donnent $4.9 \times 10^6$ t pour le dépôt actuel, $2.1 \times 10^6$ t pour les sédiments libérés vers l'aval soit un total de $7 \times 10^6$ t de matériaux déposés en 45 ans ; valeurs obtenues en multipliant les volumes par le poids volumique sec moyen (1.3 t/m$^3$).

IV. NATURE ET PROPRIÉTÉS DES DÉPÔTS :

1) Localisation des prélèvements
Trois prélèvements sont étudiés plus précisément ici (fig. 3). Les prélèvements 1 et 2 sont situés en rive gauche de la Romanche.

Le prélèvement 1 est localisé en rive droite du nouveau lit du torrent de la Pisse. Celui-ci a été creusé à la suite d'une dérivation rendue nécessaire pour la protection des travaux au pied du barrage. Ce torrent a dégagé une couche fraîche continue à travers le remplissage (fig. 6).

Le prélèvement 2, situé au pied du delta du torrent de la Pisse, correspond au dépôt entre les vidanges de 1980 et 1981. Il recouvre un niveau de "mud-cracks" datant de la vidange du printemps 1980 (fig. 7).

Le prélèvement 3 est situé en rive droite de la Romanche, en bordure d'une terrasse dégagée par la vidange de 1981.

2) Description des sédiments
Les dépôts observés dans la retenue du barrage du Chambon montrent une succession de lamines généralement silteuses et noires. Quelques niveaux très fins (rarement plus de 3 mm d'épaisseur) se distinguent par leur teinte plus claire. Quelques rares lamines ont une teinte rouille.

Des horizons grossiers (sableux à graveleux) sont observables au niveau de certains prélèvements. On peut les attribuer, suivant leur localisation, soit aux niveaux de formation du delta du torrent de la Pisse, soit à des zones de chenalisation de la Romanche lorsque le lac est en eau. Cette chenalisation est très active durant toute la période de turbidage.

L'épaisseur des lamines est variable de 0,5 mm à plusieurs dizaines de cm. L'épaisseur moyenne est d'environ 1 cm.

Il n'est pas possible de mettre en évidence par une simple observation une rythmicité saisonnière telle qu'on peut l'observer dans les varves classiques définies par De Geer (un doublet de lamines par année). Dans le cas du Chambon, chaque lamine semble être le résultat d'un épisode climatique particulier de la vie du lac.

Le prélèvement 2 représente la sédimentation de mai 1980 à janvier 1981. Il est significatif de l'abondance des dépôts successifs : 17 échantillons ont pu être séparés du fait d'un lit silteux permettant leur découllement. À l'intérieur de chacun d'eux des lamines ont pu être comptées sans pouvoir les séparer. On aboutit à un total de 54 lamines sur 68 mm (fig. 7).

3) Analyses minéralogiques
La composition minéralogique est déterminée par diffractométrie aux rayons X. Elle aboutit à une détermination semi-quantitative (la calcite étant considérée comme égalon interne).

Les résultats montrent une relative homogénéité des échantillons : 6 à 19 % de Chlorite, 16 à 36 % d'Illite, 3 à 8 % de Feldspaths potassiques, 3 à 9 % de Plagioclases, 8 à 31 % de Calcite, 21 à 43 % de Quartz. Une comparaison avec la composition minéralogique des roches mères du bassin-versant est actuellement en cours.

4) Analyses granulométriques (fig. 8)
Les fractions supérieures à 63 µ ont été tamisées. Les fractions inférieures ont été étudiées à l'aide d'un "Coulter Analyser".

VII.31
Figure 8 : Granulométrie des prélèvements 1, 2 et 3 : courbes (en poids cumulés) et diagrammes triangulaires (argile-silt-sable).
Les résultats sont présentés selon la classification de Wentworth.

L'étude de toutes les courbes granulométriques en poids cumulés et des histogrammes de fréquence simple fait apparaître une plurimodalité. Le mode principal se situe généralement entre 5 et 7 phi. Cette plurimodalité met en évidence la diversité des apports : diversité dans l'espace, du fait de nombreux petits affluents perturbant la sédimentation lacustre, diversité dans le temps, du fait de l'alternance saisonnière et des épisodes orageux, diversité de la nature des matériau, du fait des divers faciès pétrologiques rencontrés dans le bassin versant.

Que ce soit en diagramme triangulaire (pôles sable, silt et argile) ou en courbes granulométriques en poids cumulés, on met en évidence les faits suivants :

- prélèvements 1 : 24 analyses ont été réalisées. Elles se répartissent en 3 ensembles, aussi bien sur les courbes cumulatives que sur les diagrammes triangulaires.

L'ensemble A contient plus de 11 % d'argile et moins de 10 % de sable.
L'ensemble B contient jusqu'à 8 % d'argile et moins de 36 % de sable.
L'ensemble C contient jusqu'à 6 % d'argile et moins de 36 % de sable.

Cette diversité s'explique par la situation du prélèvement. On est en présence de 3 sources de sédiments, d'une part le torrent de la Pisse, qui forme en ce lieu un delta d'où résultent des dépôts grossiers, d'autre part la Romanche qui charrie des matériaux plus fins et enfin le lac en lui-même où les particules argileuses décantent.

Ces 3 sources de sédiments ne s'individuent pas respectivement dans les 3 ensembles précédemment décrits mais expliquent la diversité dans la nature des matériaux qui se reflète dans la plurimodalité.


- prélèvement 3 : Les résultats des 63 analyses se groupent dans un ensemble où se distinguent 7 éléments. Les courbes cumulatives matérialisent un faisceau. Si l'on considère le diamètre moyen des particules, 4 courbes s'ont écarts qui correspondent à des échantillons présentant plus de 50 % de sable sur le diagramme triangulaire.

Ce prélèvement montre une assez bonne homogénéité, que soit le mode de présentation des résultats.

5) Identification géotechnique
Les premières analyses des prélèvements donnent les résultats suivants :

Le prélèvement 1 a été étudié sur sa totalité soit une hauteur de 2.10 m, ce qui correspond à la profondeur de l'ancien sol sous le delta du torrent de la Pisse.
Le prélèvement 3 a fait l'objet d'analyses sur ses 2,60 premiers mètres à partir de la surface.

La densité a été mesurée sur des échantillons du prélèvement 3 au carottage de 35 mm de diamètre. On obtient un poids volumique sec compris entre 1.15 et 1.39 g/cm³ et un poids volumique apparent compris entre 1.66 et 1.89 g/cm³. Cela correspond à une teneur en eau du sol saturé comprise entre 35 et 48 %. En ce qui concerne la teneur en eau naturelle (u nat.), la moyenne se situe autour de 40 %.

Pour la cohésion non drainée (cu mesurée à l'aide d'un "Fall-Cone Géonor" sur des échantillons à teneur en eau naturelle, soit un indice de saturation compris entre 0.80 et 0.96) on a :

- pour des échantillons du prélèvement 3 19 à 19 kPa pour les échantillons remaniés
19 à 70 kPa pour les échantillons intacts.

En terme de sensibilité (St, rapport du Cu remanié à Cu intact), ces résultats donnent des valeurs s'étalant de 1,2 à 6,5 (matériau peu sensible à sensible).

- pour des échantillons du prélèvement 1 (à partir de 1.16 m de profondeur) 18 à 23 kPa pour les échantillons remaniés
41 à 78 kPa pour les échantillons intacts.

Ce qui fait varier la sensibilité de 1.3 à 4.3 (matériau peu sensible à sensible).

Les essais in situ ont montré que la résistance totale (Rt) mesurée au pénétromètre varie entre 50 kPa et 1 MPa pour les niveaux argilo-sileux. On peut atteindre 7.5 MPa pour les niveaux plus grossiers. On obtient plus de 10 MPa pour le paléosol. Ces essais soulignent l'hétérogénéité verticale des sédiments. La résistivité mesurée lors du sondage électrique est de 38 Ω m.

VII.33
CONCLUSION

Les vidanges totales de la retenue du barrage du Chambon en 1980 et 1981 nous ont permis l'étude directe des matériaux déposés depuis 45 ans. Le volume total des sédiments présents après la vidange de 1981, évalué à 3.8 $10^5$ m$^3$, ne correspond pas à la totalité des matériaux arrachés au bassin versant par érosion puisqu'une partie d'entre eux s'est écchappée lors du fonctionnement normal de l'ouvrage et au cours des vidanges complètes.

En ce qui concerne le fonctionnement normal, nous ne disposons d'aucune mesure au turbidimètre sur les eaux turbinées. On peut remarquer cependant que si l'on prend une valeur de $10^{-2}$ g/l (valeur probablement très supérieure à la réalité), on obtient une masse évacuée par turbinage depuis la dernière mise en eau du barrage d'environ $1.4 \times 10^3$ t, ce qui reste négligeable devant les volumes d'alluvions sétenant sedimentés.

En considérant que les volumes évacués lors des vidanges successives correspondaient au moins à celui de la vallée entaillée par la Romanche jusqu'au niveau des plus hautes terrasses, on obtient une valeur approximative de $1.6 \times 10^6$ m$^3$. Ramenés en terme de poids de matière solide, en tenant compte d'un poids volumique sec (yd) moyen de 1.3 g/cm$^3$, on obtient le chiffre total de $7.10^6$ t, ce qui correspond à une production moyenne de 450 t de sédiments par km$^2$ et par an ou encore à une érosion de 0.25 mm/an. Localement, dans les zones schisteuses, cette érosion dépasse certainement le cm par an.

Le transport des matériaux solides s'effectue de manière prépondérante à la fonte des neiges et plus encore en été et en automne après les orages. Les charges solides des eaux arrivant dans la retenue sont très variables. En période de crue, elles doivent atteindre une dizaine de g/l. Lors de la vidange de 1981, le maximum de charge solide des eaux quittant la retenue était de environ 30 g/l. La valeur moyenne mesurée en comparant la masse des sédiments produits et la quantité d'eau ayant transité par le lac du Chambon depuis sa mise en eau ($1.4 \times 10^5$ m$^3$) donne seulement 0.43 g/l ; ce qui montre bien le caractère très discontinu du remplissage. Ceci est confirmé par l'observation des lamines dont l'épaisseur varie de 0.5 mm à plusieurs dizaines de cm.

Les analyses granulométriques confirmait la discontinuité des apports qui toutefois n'apparaît pas dans les analyses minéralogiques.

Les résultats précédents doivent être complétés prochainement par ceux qu'apportent le dépouillement complet des analyses granulométriques et minéralogiques et les mesures systématiques des propriétés physiques et de la cohésion, dans le but de mieux définir le phénomène de compaction des sédiments lacustres actuels.

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SILATION OF DAL LAKE—SOME CAUSES AND REMEDIES

ENVASEMENT DE DAL LAKE—DES CAUSES ET DES REMEDES

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Geological Survey of India
Lucknow, India

ABSTRACT

Dwindling of Dal lake in J & K State of India is a matter of great concern to all. It is alarming to note that the lake has retreated from 32 sq. km to 8 sq. km area. Sedimentation of about 3,00,000 cu. metre/year is the main cause responsible for reduction in the size of the lake. These sediments comprise about 8% fine sand with clay, 89% fine sand and 3% medium grained sand. This siltation is governed by geology, topography, climate and vegetation in the catchment area. Based on these parameters certain remedial measures for example construction of check dam, afforestation, prevention of overgrazing and manual dredging have been suggested to check the siltation in Dal lake.

ABSTRACT

Diminution de Dal Lake à l'Etat de Jammu et Kashmir en Inde est une affaire d'inquiétude sérieuse pour tout le monde. Il est alarmant à remarquer que l'étang s'est retiré de 32 km² à une région de 8 km² seulement. Sédimentation de à peu près 3,00,000 m³/an est la cause principale responsable pour réduction à la dimension de l'étang. Ces sédiments comportent à peu près 8% de sable raffiné avec argile, 89% de sable raffiné et 3% de sable moyen. Cet envasement est gouverné par géologie, topographie, climat et végétation dans la surface de captation des eaux. Foncé sur ces paramètres certaines mesures réparatrices par exemple construction d'un barrage de contrôle, reboisement, empêchement de sur-écorchure et dragage manuel ont été suggérés pour arrêter l'envasement à Dal Lake.

INTRODUCTION

Dal lake in India attracts about 3,000 tourists per day during summers from all over the world. Dal lake is connected with the river Jhelum through a gated canal from which a controlled inflow of water is maintained. The lake measures about 5.2 km in length and 1.5 km in width and provides livelihood to 1800 houseboat owners and 6000 fishermen who do fishing of 1100 tons per annum. Shocking but true, the existence of Dal lake is in peril and if the present environment persists the lake may vanish in the near future.

CAUSES OF SILTATION

It is a multi facet phenomena. The inflow of sediments coupled with dumped sewage help extensive development of various aquatic flora. Decay and settlement of this flora along with accumulation of sediments and garbage from various encroachments help the shallowing of the lake.

1. ENCROACHMENTS

Extensive encroachments during the last 30 years have resulted in diminishing the Dal lake area from 32 sq. km (approx. 8 km x 4 km) in 1947 to merely 8 sq. km (approx.
5.2 km x 1.5 km) in 1979. About 100 establishments, 20000 residential buildings, floating gardens on 200 hectares and cultivation on 400 hectares have also helped in reducing the Dal lake area.

2. Sewage disposal

The Dal lake has become a dumping ground for refuse of city. Various chemicals, fertilisers and pesticides from nearby agricultural fields, orchards and forests together with the animal droppings from Dachigam sanctuary add to the shallowing of the lake bed. About 1800 houseboats in the lake provide boarding and lodging to the tourists (Fig. 1) due to which about 95 tons of sewage gets deposited in the lake bed every year.

Fig. 1. Photograph of a part of Dal lake.

3. Floral growth

Growth of numerous weeds and water bearing plants are causing great damage to the lake. Though every day 20 to 30 boat loads of weeds are collected manually but they grow so fast that it is difficult to eliminate them. The problem of this botanical growth is equally felt in Khajjar and Rewalsar lakes near Chamba in Himachal Pradesh, Usmania lake of Hyderabad, Michola lake, Swaroop lake and Boodh Talai lake in Rajasthan. These aqueous plants first develop in marshy, dirty water; shift in clean water by winds and grow rapidly by absorbing more and more water. These have been found to contain about 95% water by weight. They develop twice within a week and 100 times in about six weeks thereby reducing the volume of water body.

Floating island is a common feature causing reduction of the Dal lake. Since 1914 to 1964 an area of about 200 hectares has been converted into floating islands on the western fringe of Dal lake (Fig. 2). The development of floating islands takes place in four stages. The analysis of soil samples from these floating islands show certain interesting correlations between the physical and chemical characteristics and their developmental stages. The observations indicate that the whole process of the formation of island is primarily a soil forming process in a basin which is very much akin to one taking place elsewhere in nature (Table 1).

Table 1. Developmental stages of floating island

<table>
<thead>
<tr>
<th>Stages of Development</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Thickness</td>
<td>cm</td>
<td>cm</td>
<td>cm</td>
<td>cm</td>
</tr>
<tr>
<td>cm</td>
<td>15-23</td>
<td>23-46</td>
<td>46-61</td>
<td>Stabilises island with ground</td>
</tr>
<tr>
<td>2. Physical properties of substratum (dry weight in pctg.)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Clay</td>
<td>20.0</td>
<td>15.0</td>
<td>13.5</td>
<td>11.0</td>
</tr>
<tr>
<td>b. Silt</td>
<td>24.5</td>
<td>15.5</td>
<td>24.0</td>
<td>20.0</td>
</tr>
<tr>
<td>c. Fine sand</td>
<td>50.0</td>
<td>65.0</td>
<td>53.0</td>
<td>62.0</td>
</tr>
<tr>
<td>d. Coarse sand</td>
<td>0.7</td>
<td>0.28</td>
<td>2.0</td>
<td>2.2</td>
</tr>
<tr>
<td>3. Reaction (pH)</td>
<td>7.3</td>
<td>7.6</td>
<td>7.9</td>
<td>7.9</td>
</tr>
<tr>
<td>1. P₂O₅</td>
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<td>0.008</td>
<td>0.001</td>
<td>0.006</td>
</tr>
<tr>
<td>2. K₂O</td>
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<td>0.05</td>
<td>0.055</td>
<td>0.06</td>
</tr>
<tr>
<td>3. N₂</td>
<td>0.031</td>
<td>0.3</td>
<td>0.25</td>
<td>0.31</td>
</tr>
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<td>4. C</td>
<td>1.1</td>
<td>2.5</td>
<td>1.50</td>
<td>1.4</td>
</tr>
<tr>
<td>5. C/N</td>
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<td>8.0</td>
<td>6.0</td>
<td>4.5</td>
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<td>6. MgO</td>
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<td>0.55</td>
<td>0.80</td>
<td>1.2</td>
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<td>7. CaO</td>
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<td>0.85</td>
<td>2.0</td>
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<tr>
<td>8. P₂O₅</td>
<td>2.6</td>
<td>2.88</td>
<td>4.50</td>
<td>5.50</td>
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<tr>
<td>9. L.O.I.</td>
<td>9.00</td>
<td>30.00</td>
<td>12.00</td>
<td>12.50</td>
</tr>
<tr>
<td>10. Silicate oxide</td>
<td>9.00</td>
<td>7.50</td>
<td>13.50</td>
<td>15.2</td>
</tr>
</tbody>
</table>

4. Geodynamics

The catchment area of Dal lake is approximately 180 sq. km and the rocks exposed around it are given in Table 2. Dal catchment area besides hard rocks includes terraces, alluvial fan, talus cones and rock debris also. The estimated siltation to the order of about 300,000 cu.m./year is causing irreparable damage to the Dal lake. The soil derived from the volcanic rocks is very shallow. It is light sandy loam with enough clay. Their water holding capacity is very poor. By constant scuffling of cattle the surface becomes dusty making the soil vulnerable to easy transportation and
Fig. 2. SKETCH PLAN OF LAKE CATCHMENT
siltation.

Table 2. Lithology of Dal catchment

9. Scree and alluvium (Boulders, pebbles, terraces and river borne sand)
8. Karewas (Sandy, silty and pebbly conglomerates)
7. Jurassics (Limestones with shales)
6. Triassics (Mainly limestone)
5. Permo-Carboniferous (Sandstone, clay shale and limestone)
4. Panjal trap (Quartz-basalt)
3. Agglomeratic slates (Mainly slates)
2. Slate series (Quartzites, slates and phyllites)
1. Granites

The main drainage of the area is Dagwan nala. It flows through traps, limestones, calc-shales, sandstone and alluvium. The catchment area experiences heavy snow fall which help to disintegrate the country rocks. The rains coupled with strong winds and steep gradient activate the erosion and subsequent siltation.

Siltation to the order of 79,000 cu.m./year has been observed from Dagwan nala nearTelhal. It comprises 8% clay and very fine sand measuring less than 0.075 mm in size, 88% fine sand measuring 0.075 mm to 0.425 mm in size and 3% medium grained sand measuring 0.425 mm to 2 mm in size. Siltation has left only 2.74 m water body by 1981 in the lake, with deepest water level being 1580.69 m and average water level standing at 1583.43 m (Fig. 3).

Considering Dagwan nala as a model for silt load calculation for the entire Dal lake catchment the silt which would enter the Dal lake works out to the order of 30,000 cu.m./year. From the study of the Dal catchment it is seen that in the overall catchment the soft rock formation comprising karewas and alluvium is of the order of 40% whereas the same in Dagwan nala is 18% therefore, it is reasonable to assume that the actual silt yield from the catchment as a whole will be more than 3,00,000 mentioned above. Based on the present volume of the water body of Dal lake and the rate of siltation worked out, the life of the Dal lake is 36 years. If Fourniers formula is utilised for silt yield per annum it is worked out to the order of 2,14,65,000 cu.m. which is seven times more than the actual observation. If the silt yield load is calculated by this formula the life of the lake is only 5 years which does not appear realistic. The reason of it could only be explained that the geological parameters, i.e. the hard/soft rock types are not taken into consideration.

<table>
<thead>
<tr>
<th>H.F.L.</th>
<th>NORMAL W.L.</th>
<th>MINIMUM W.L.</th>
<th>DEEPEST W.L.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1584 m.</td>
<td>1583 m.</td>
<td>1582.5 m.</td>
<td>1580.6 m.</td>
</tr>
</tbody>
</table>

Fig. 3. Generalised section of Dal lake deposits.

Preventive Measures

For protection of the Dal lake recently a 'Lake Development Project' of Rs.22 crores has been drawn which is expected to be completed within 8 years and at present about 500 men are engaged in it. Under this project various encroachments are to be evicted. A foreshore around the lake is being raised for house-boats so that the centre of the lake remains clean. Other measures for example the construction of underground sewage tunnel is being considered with its outlet to a suitable downstream point where a sewage disposal plant could convert the refuse into the fertiliser.
Regarding siltation problem of the lake the inflow of the silt depends mainly on the condition of the catchment. As sediments originate from the watersheds the obvious point is to prevent the sediments from being carried into the lake. The study of sedimentation should therefore, include climatic conditions, rainfall, geology, land use pattern and vegetal status.

It is worth mentioning some old water bodies existing for long without siltation because of their vegetal status. The Pakhal lake, Ramappa lake, Shangaram tank and Cumbum in Andhra Pradesh may be cited as examples. It is observed that the forested area has played a great part in lengthening the lives of Pakhal lake and Ramappa lake because it has helped in controlling the erosion of soil by keeping it intact and minimizing the sediment flow into the lake.

A good vegetative cover on the watershed will be the best protection for sediments and it will ensure the longer life of Dal lake. It will protect the land against all forms of deterioration. Sheep's dense grazing is a very common phenomenon of this part of the country and should be discouraged as it renders the soil vulnerable to erosion. Besides providing the vegetative screen (afforestation) the deforestation should be banned under law in whole of the catchment area.

Depending upon the topography and other local conditions prevailing in the area the engineering proposals like check dams, contour bund, gully plugging, diversion or by-pass channels can be attempted to control the siltation.

The check dam reduces the velocity of stream flow and thus unload the sediments. As the life of the check dams is small they should be treated as temporary measures and other structures may be considered simultaneously. Contour bund, trenching and terracing are important methods of controlling erosion on the sub-horizontal gradients. By these method the hill side is split into small compartments on which the rain is retained, run-off is modified as the soil erosion is checked.

Dagwan nala emerges from the eastern hills with a very narrow gorge section. There is no storing place available as reservoir for accumulated water and sediments except at Sarbal. Various small check dams on different streamlets can be constructed to control the siltation partly. Stored water can be utilised for irrigation also. An earthen dam across Dagwan nala near Sarbal is seen feasible but its techno-economic feasibility is to be studied in detail and taken into consideration. Dagwan nala drains about 30% of the total catchment area of Dal lake, therefore, any check dam constructed on Dagwan nala and its tributaries will check the sediments from only 30% of the area. The catchment of Dagwan exposes the following strata percentage-wise:

1. Panjal volcanics 56%
2. Alluvium 16%
3. Jurassic 15%
4. Triassic 10%
5. Permo-Carboniferous 3%

Geologically and percentage-wise distribution of the various rock types in Dal lake catchment is given in Table 3.

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Lithology</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Granites</td>
<td>1%</td>
</tr>
<tr>
<td>2</td>
<td>Slates and quartzes</td>
<td>4%</td>
</tr>
<tr>
<td>3</td>
<td>Aqglomeratic slates</td>
<td>1%</td>
</tr>
<tr>
<td>4</td>
<td>Panjal traps</td>
<td>46%</td>
</tr>
<tr>
<td>5</td>
<td>Permo-Carboniferous</td>
<td>1%</td>
</tr>
<tr>
<td>6</td>
<td>Triassic limestone</td>
<td>3%</td>
</tr>
<tr>
<td>7</td>
<td>Jurassic</td>
<td>4%</td>
</tr>
<tr>
<td>8</td>
<td>Karwas</td>
<td>15%</td>
</tr>
<tr>
<td>9</td>
<td>Alluvium</td>
<td>25%</td>
</tr>
</tbody>
</table>

It is seen from the above Table that out of 180 sq. km area around Dal lake, about 46% is covered by Panjal volcanics and 40% is covered by unconsolidated Karwas and alluvium. Rest 14% is covered by other rocks. Even after constructing a check dam 60% of the total catchment and particularly 40% soft and vulnerable sediments remains uncontrolled. Therefore, the contour bund, gully plugging, terracing and trenching may also be considered. The contour bund along lake's periphery is to be diverted to Jhelum river in downstream. However the excavation of a trench may involve the socio-economic problems because of dense population.

The rains followed by quick run-off bring the sediments into Dal lake basin from Karwas country rocks and alluvium through various gullies and rills. These already deposited sediments in Dal lake are to be removed by manual excavations and/or mechanical dredging. Keeping in view that floating islands already developed can not be now removed, check dam is not a total success, only dredging comes to rescue for...
keeping the lake alive for a longer time. The developing of floating gardens as on western fringe of Dal lake is to be totally banned so that no more lake area could be converted into dry land.

CONCLUSIONS

The discussion has shown that the area of Dal lake is increasingly reduced and the various parameters responsible for it are the geodynamics processes, encroachments, sewage disposal and development of the floating islands etc. For prevention of siltation in Dal lake it is desirable that these parameters should be restricted to a minimum. Check dam is recommended near Sarbal across Dagwan nala and on its various other tributaries to check the siltation. Overgrazing in the Dal catchment area is to be controlled strictly. Deforestation should be discouraged and afforestation should be encouraged. Already accumulated sediments are to be removed by manual excavation and/or mechanical dredging.

ACKNOWLEDGEMENT

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CONSIDERATIONS SUR LA SEDIMENTATION DANS LES BARRAGES—FORMATION DES DEPOTS, EXEMPLES, MOYENS DE LUTTE

CONSIDERATIONS ON THE DAM SEDIMENTATION—DEPOSITS FORMATION, EXAMPLES, MEANS TO FIGHT

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Cemagref de Paris
Antony, France

ABSTRACT
Après avoir souligné l'importance de la sédimentation dans les barrages, l'auteur traite du processus classique de la formation du plateau sous-lacustre et donne quelques exemples d'alluvionnement dans différents pays. Il fait part, ensuite, des principaux moyens de lutte d'ordre curatif dans le cas de dépôts déjà formés et dans le cas de dépôts en cours de formation.

ABSTRACT
After emphasizing the importance of the dam sedimentation problem, the author deals with the classical mechanism of the underwater slope formation and gives examples of dam sedimentation in different countries. He then reviews the main means (those of the curative type only) to fight against already existing deposits as well as against gathering deposits.

On peut classer, schématiquement, les effets de l'environnement sur les barrages en effets qui relèvent de la matière vivante ou de la matière inanimée.

- Biologie : groupements humains (limitation du niveau de l'eau du fait d'habitations, contraintes dues à la navigation, à l'esthétique ...), animaux (échelle à poissons, débits réservés à l'aval ...) végétaux (prolifération des jacinthes d'eau, action de débris végétaux voire de troncs d'arbres ...).

- Matière inanimée : évaporation, écarts importants de température, avalanches, mouvements de terrains, sédimentation dans le réservoir, lithologie, qualité de l'eau, (attaques des bétons ordinaires par l'eau séléniteuse, pepition des colloïdes) etc...

Parmi la seconde catégorie d'effets, la sédimentation dans les barrages est le processus le plus courant. Elle fait partie des causes de réduction de la capacité des réseaux de réseaux (comme les fuites, l'évaporation, les mouvements de terrains dans la cuvette ...) et en est la plus complexe et la plus difficile à maîtriser. Cette sédimentation est sensible non seulement dans les régions arides, caractéristiques à ce point de vue (saisons sèches marquées, pluies violentes, terrains meubles secs et pauvres en végétation ...) mais encore sous des climats tempérés en certains reliefs accusés (le lac de Chambon sur la Romanche dans les Alpes françaises possède un niveau de vase à quelque 18 m au-dessus de la vidange de fond, devenue inutilisable). L'intensité d'une telle sédimentation varie d'un minimum théorique (cas idéal d'un barrage dont on détourne par dérivation les débits d'hiver et de crue en région tempérée) à un maximum qui, à notre connaissance, est illustré par le barrage de Laguna (Colorado,
U.S.A.) dont la retenue d'un volume de 25. 10^6 m^3 fut comblée en une année par la San Juan River.

L'Impact de la sédimentation dans les barrages peut se répercuter aussi bien à l'échelon national. Ainsi M. THEVENIN (in J. VALEMOIS et al. 1975) souligne qu'en 1957 les barrages d'Algérie, d'une capacité totale de 900. 10^6 m^3 avaient accumulé près de 200. 10^6 m^3 de vase et que l'on pouvait prévoir pour la fin du siècle une réduction des 2/3 de la capacité totale.

On voit toute l'importance de ce problème. Nous évoquerons ci-après la formation des dépôts, puis quelques exemples et, en troisième lieu, la lutte contre l'envasement.

1 - FORMATION DES DÉPÔTS.

Les apports solides, on le conçoit facilement, sont fonction de :
- la climatologie : régime des pluies (arrachement des particules) et de la température (sensibilisation du sol par dessèchement)
- la couverture végétale au rôle anti-érosion
- la lithologie (nature du terrain)
- la morphologie (état de la surface, relief).

Après l'érosion et le transport (conditionné par la compétence du cours d'eau) les sédiments arrivent dans la retenue et s'y déposent selon un mécanisme à peu près constant.

Mécanisme.

Freiné par l'eau du réservoir, le cours d'eau voit sa compétence diminuer et les sédiments se déposent.

Ils se déposent dans la queue de la retenue et il se forme, à partir de là, un plateau sous-lacustre (souvent appelé delta) à faible pente se terminant, vers l'aval, par un front à forte pente. D'une manière très générale ce plateau est constitué en majorité de matériaux charriés dans les grandes retenues alors que la proportion des matériaux en suspension est plus forte dans les petites retenues.

D'une manière générale aussi, la granulométrie est naturellement plus grossière en amont du delta mais c'est souvent, en vérité, plus compliqué car la distribution des dépôts par granularité dépend aussi du niveau d'eau de la retenue et du cheminement des sédiments dans telle ou telle partie de la retenue. Il se produit en outre des érosions du delta lors de l'abaissement du plan d'eau.

Dans le cas théorique d'une retenue prismaticque, pour un cote Z du niveau d'eau dans la retenue, G. REMENIERAS et G. BRADEAU (1951) démontrent par calcul (formule de CHEZY, formule de MEYER-PETER) le processus de formation des dépôts dans le réservoir, à partir de rognons d'exassement. Ils chiffrent ainsi le fait que lorsque le débit liquide augmente seul il y a dégagement de la retenue alors qu'il y a engraissement lorsque c'est le débit solide qui augmente seul. Mais là, admettent les auteurs, la réalité est plus complexe, ainsi que nous venons de le signaler à propos de la distribution des dépôts.

Cette distribution se complique encore du fait de l'existence des courants de densité (under flow) constituées d'eaux troubles (et plus denses par concentration des matériaux fins) au sein des eaux claires du lac. Ils cheminent aussi bien au-dessus du fond du lac que sur le fond et leur étude présente un intérêt considérable comme nous le verrons plus loin.

Ces dépôts, d'autre part, se consolident d'une manière très variable. Dans le cas du barrage de Cheurfas (Algérie), par exemple, G. DROUIN et al. (1951) soulignent le fait que la vase dans ce réservoir n'était toujours pas consolidée après 70 ans d'existence. Dans le cas du barrage de l'Oued Fodda (Algérie) la vase, d'une puissance de plus de 30 m, se consolidait rapidement sous l'eau. Cette consistance peut d'ailleurs être telle qu'il devient nécessaire en certains barrages de l'attaquer à la pioche pour la désagréger. Quand on sait que la vase séchée est normalement très peu dense - comme nous l'avons constaté nous-mêmes lors d'une vidange périodique du barrage de Guerlédan en Bretagne - on admet facilement que les phénomènes physicochimiques de consolidation de la vase relèvent de paramètres assez différents et dont la pondération est très variable.

* c'est-à-dire transportés par soulèvement ou salutation sur le fond du lit des cours d'eau.

VII.42
2 - EXEMPLES DE SÉDIMENTATION.

La jeunesse de leur relief, la nature de leur terrain (roche calcaire, morainée) et de leur climat (gel - précipitations) font des Alpes françaises une région privilégiée de la sédimentation, à plus fort degré en tous cas que les Pyrénées, dans leur ensemble. Ces dernières peuvent cependant présenter certaines années une érosion spécifique plus élevée (aboulement, changement de lit ?). Le massif Central français vient en troisième lieu, ce qui n'exclut pas des anomalies locales à érosion spécifique élevée.

A une autre échelle, les causes de variation dans la sédimentation peuvent venir du fait que les facteurs n'interviennent pas toujours dans le même sens : G. REMENIE RAS et G. BRAUDEAU (1951) observent "... la couverture végétale intervient pour protéger les terrains contre l'érosion mais les observations faites au réservoir de la petite Rhue en France montrent que lorsque le débit solide est faible et la forêt très étendue, les débris végétaux peuvent représenter une notable fraction des dépôts dans les petites retenues..." Nous avons fait la même observation à propos des sédiments déposés dans la fosse d'un bassin versant expérimental sur schistes briovériens (France).

La Tchécoslovaquie offre l'exemple d'un même bassin qui possède deux régimes d'alluvionnement suivant la localisation des pluies et la nature de la roche mère (V.L. HOLOCEK - 1951). La rive droite du Vah moyen est principalement constituée de moraines, de fluvio-glaciaire et de flysch (grès calcaires, schistes marneux et argileux) tectonisés et désagrégés, formant par conséquent de puissantes masses de débris éminemment terrigènes saturant les affluents ; la rive gauche, aux affluents non saturés d'alluvions, comprend des granites et des terrains cristallins plus ou moins enrobés, par nappé de charriage, de calcaires et dolomies souvent karstifiées.

En Égypte, le volume au barrage d'Assouan a été calculé en fonction de l'alluvionnement probable. Sur les 164 km² de capacité, 30 km³ sont réservés au dépôt des limons. Le taux annuel de sédimentation de ces limons représenterait 0,002 % des 30 km³, soit une durée de vie, pour le barrage de quelque 500 ans (J.Z KINANY et al. 1973). C'est là une solution au problème de sédimentation dans les barrages, quand elle est possible (relief, prix de revient).

Les prévisions ne sont pas toujours conformes à ce qui se passe par la suite. Ainsi J.P. GUPTA et al. (1981) signalent à propos du barrage de Matatila que l'indice de sédimentation réalisé (38.10³ m³/an/100 km²) était trois fois l'indice supposé au cours du projet.

La coupe de la retenue du barrage d'Iwashimizy donnée par H. ASADA (1973) et qui rend compte du profil des dépôts depuis 1960 jusqu'à 1967 est assez instructive. Elle fait apparaître deux choses :


- cette stratification chronologique sensiblement parallèle à la pente du lit est doublée d'une stratification granométrique à peu près horizontale dont la strate I la plus profonde correspond à du limon et du sable fin et la strate IV à du sédiment plus lourd avec tous intermédiaires avec les strates II et III. Le granoclassessement est donc assez constant dans le temps.

3 - LA LUTTE CONTRE L'ENVASEMENT.

Cette lutte ressort à deux grands groupes de mesures :

- d'ordre préventif : c'est la mesure de l'érosion, vaste domaine tant sur le plan pratique (procédés de mesures) que théorique (modèles mathématiques ou physiques de prévision - voir entre autres le Congrès de New-Delhi 1981, de l'Association Internationale de Recherches Hydrauliques) et la protection des sols (végétation, banquettes ...). Eventuellement la dérivation du cours d'eau pendant les périodes de crues.

- d'ordre curatif : après l'alluvionnement (réduction des dépôts) et pendant l'alluvionnement dans la cuvette.

* Au sens littéral du mot (qui donne de la terre) et non pas au sens habituellement accepté en géologie, sens qui est un remarquable solélisme prétendant signifier "qui vient de la terre".
Nous n'évoquerons ici que les mesures d'ordre curatif.

3.1. Après l'alluvionnement.

Parmi les procédés de lutte utilisés après l'alluvionnement, le dragage à la drague succeuse est relativement onéreux mais réalisable à l'échelle quasi-industrielle ainsi que cela a été montré depuis longtemps en Algérie (G. DROUHIN et al. 1951). Compte tenu de l'importance du phénomène dans ces régions (cf plus haut M. THEVENIN) et du fait que les sites économiquement favorables à la construction de nouveaux barrages sont rares, des moyens de lutte même onéreux peuvent être rentables. La méthode tablait sur le fait que la consommation d'eau était de 4 à 7 fois le déblai dragué, les 3/4 de l'eau était d'ailleurs récupérés pour l'irrigation dans des casiers à décantation.

On peut encore réduire les dépôts par effet de chasse d'eau, l'ouverture des vannes formant un courant qui entraîne les alluvions sédimentées. A. DIACON et al. (1973) citent le cas d'une retenue de Roumanie qui après rétablissement du courant d'eau naturel ! s'est trouvée délestée, en une seule crue, de 25% de ses sédiments. Au lieu du cours d'eau naturel, lorsqu'il ne peut remplir ce rôle, on peut faire agir un barrage auxiliaire amont. Les lâcheres font progresser les matériaux submergés jusqu'au parement amont du barrage principal (transport-approche) puis, les vannes de dégravesment étant ouvertes, de nouvelles chasses évacuent les dépôts (transport-dégravesment).

Lorsque cette opération n'est pas possible, il faut vider la retenue et laisser les vannes ouvertes à la première crue (ou pendant un temps assez long, méthode de purge dite espagnole qui laisse le cours d'eau reprendre son naturel). D'autres procédés encore ont été efficaces, tel l'envoi de puissants jets d'eau, tel la dénivellation faisant tomber l'eau de lavage de la cote majeure au niveau des dépôts afin de les désagréger (barrage du Fergou, Algérie). On peut parfois être conduit à dégager les marnes par jets d'eau sous pression ou air comprimé. Pour le dégravesment de la galerie de Motty (France), il a fallu construire une galerie de décommatage et la déboucher par 400 kg de dynamite.

On peut encore utiliser des bennes pour vider les petites retenues.

Lors de ces opérations le risque existe de colmater le cours d'eau en aval du barrage et il faut y penser. J. VALEMBOIS et Al. (1975) traitent de ce problème : un plan de dragage ou de chasse des retenues peut être établi à partir d'études sur la capacité de transport des rivières, basées sur la nature et les propriétés physiques des matériaux sédimentés dans la retenue et sur des considérations théoriques et des modèles hydrauliques.

3.2. Pendant l'alluvionnement.

L'utilisation des courants de densité (under flow) est un moyen de lutte efficace. Ces courants de densité, veines issues du courant boueux freiné par la retenue, contenant des matières colloïdales et siltées, s'individualisent du fait de leur plus grande densité (1,100 à 1,300 dans un barrage expérimental H. DUQUENNOIS - 1951), de leur salinité et de leur différence de température. Parfois très coloriées, les veines coulent en tous niveaux (surface, en amont de la retenue, entre deux eaux ou au fond près du barrage). L'étude du barrage expérimental d'El-Ouldja permet d'intéressantes mesures in-situ et montre que l'évacuation de ces courants par des vannes était très réalisable. La formation de ces courants, leur direction et la disposition des vannes doivent, dans chaque cas, faire l'objet d'études précises.

Une méthode semblable consiste à soutirer simplement la vase du fond à l'aide de vannes, à condition que le dépot soit encore liquide (peut de 3 à 4 m³ d'eau pour récupérer 1 m³ de capacité, d'après H. DUQUENNOIS - 1951).

Dans le même ordre d'idées, la méthode de chasse à niveau bas consiste à maintenir un bas niveau de la retenue, au moyen de grosses vannes, pendant une période de fort débit du cours d'eau dans les retenues.


On peut encore, comme à Serre-Ponçon, avoir des ouvrages de prise d'usine et de vidange de fond à section très large, avec "injecteurs de turbulence".

D'autres actions pendant l'alluvionnement sont possibles. Ainsi la construction de barrages de sédimentation (débris dams) en amont d'un réservoir à protéger. Cette idée, qui n'est pas nouvelle, fut notamment émise par A. BRIVES en 1925 (in
Y. GOURINARD - 1952. Consulté sur l'enva-
sement probable du barrage de l'Oued Fodda
(Algérie), ce géologue note que le bassin
versant correspondant comptait 55 % de
schistes crétacés esquilleux durs à végé-
tation forestière, mais encore 45 % de
terrains tertiaires surtout marneux qui
seraient en l'occurrence les grands respon-
sables d'un envasement évolutif. De fait,
85 % en poids des vases à l'emplacement
intéressé étaient d'origine tertiaire. La
géologie inspirait directement cette idée
de barrage à sédimentation, qui pouvait en
l'occurrence être placé à la limite crétacé-
tertiaire.

Un autre procédé consiste à former des
plages d'épandage naturelles ou artificiel-
les (dans ce dernier cas par gabions qui,
réduisant la vitesse du courant, permettent
un dépôt "prématrué" en aval du barrage).
Cette solution n'est pas efficace dans le
cas de matériaux uniquement colloïdaux.

La surélévation n'est pas un procédé de
lutte "contre" l'alluvionnement mais
plutôt "avec" (barrage du Hamiz en Algérie.
G. DUROZQY 1952). La topographie cependant
doit être telle que l'opération soit réa-
lisable et d'autre part il faut tenir
compte, dans les pays chauds et arides, d'
un risque d'autophagie, l'augmentation de
surface qui correspond à la surélévation
entraînant une évaporation accrue.

Un autre moyen de lutte "avec" consis-
té à prévoir un certain volume de la réser-
vie sacrifié au dépôt de matière (dead sto-
rage) comme nous en-donnons l'exemple au
§ 2.2. (barrage d'Assouan).

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AN APPROACH TO PREDICTING AND EVALUATING THE SHORELINE PERFORMANCE OF MAN-MADE LAKES

UN ACCES DE PREDICTION ET D'ÉVALUATION DU FONCTIONNEMENT DES RIVES DE LACS ARTIFICIELS

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ABSTRACT

This paper describes the author's experience in developing a reliable procedure for predicting and evaluating reservoir shoreline performance in British Columbia, Canada.

A model program typically involving three phases of investigation and study is followed. This phased program is compatible with the typical engineering schedule for a hydroelectric development. The methodology used involves initial classification of the shoreline with respect to predicted nature, magnitude, location, and probability of occurrence of bank instability. The classification format readily permits comparison with existing conditions. Later phases of the program involve computing post-failure dynamics (i.e. velocity-displacement aspects) of potential problem slide areas to be utilized in hydraulic model tests to predict slide generated wave characteristics.

The safeline concept now applied to most reservoirs throughout the province was developed as part of the above program. This concept, described in detail in the paper, is a valid and useful tool for establishing project costs and feasibility in the early phases of investigation and for land use control and zoning in the later phases.

ABSTRACT

La présente étude décrit l’expérience acquise par l’auteur en Columbia Britannique and Canada de mise au point d’une méthode fiable de prédiction et d’évaluation du comportement des rives de réservoirs.

On suit un programme-pilote qui comprend généralement trois phases d’enquêtes préliminaires et d’études. Ce programme phase est compatible avec les plans techniques typiques d’un aménagement hydroélectrique. Selon notre méthodologie on doit établir la classification des rives suivant les prévisions de la nature, de l’ampleur, de la localisation et de la fréquence probable des phénomènes d’instabilité des berges. L’utilisation de ce format de classification permet de les comparer.
facilement à des situations réelles. Les phases ultérieures du programme traitent du calcul des dynamiques d’après-rupture (c.a.d. des aspects vitesse-déplacement) des endroits à fort risque de glissement dont on se servira dans des essais hydrauliques sur maquette pour calculer à l’avance les caractéristiques de l’ondulation créée par les glissements de terrain.

Le concept de “ligne de sécurité” que l’on applique maintenant dans la plupart des réservoirs de la province, a été mis au point dans le cadre du programme décrit ci-dessus. Ce concept, expliqué en détail dans notre étude, se révèle un outil valable utile pour l’étude du coût des projets et des possibilités dans les phases préliminaires des enquêtes, et pour le contrôle de l’utilisation des sols et pour la répartition en zones dans les phases ultérieures.

INTRODUCTION

Over the last 20 years British Columbia (Canada) has seen the creation of several large man-made lakes through the hydroelectric development of the Columbia and Peace River Basins. Some of these reservoirs were created in populated areas where subsequent shoreline instability and regression was an important factor in land use planning and a potential threat to the security of both residents and shoreline structures. Some of the questions most commonly posed were: will the shoreline regress by sliding or beaching; when, where and at what rate will it regress; how much setback from the initial full supply level should be allowed; will landslides move quickly and if so what size waves will develop; what is the probability of occurrence of landslides and will there be adequate warning?

This paper describes the author’s experience throughout this period in developing a reliable procedure for predicting and evaluating reservoir shoreline performance.

CAUSES OF SHORELINE REGRESSION AFTER FLOODING

Landslides

Flooding of the toe of an existing slide or slump area, whether it be in overburden or bedrock, will generally reduce stability provided the flooding extends above the pre-flood water table. This results largely from the reduction in effective stresses at the toe of the slope. At the same time the forces tending to cause movement are relatively little affected by partial flooding, and thus overall stability is reduced. This mechanism is illustrated in Fig. 1. The resistance to shearing along pre-existing slide planes can be assumed to be close to the residual value, and the rate of movement of any reactivated slide would probably be slow. However, rapid movements of the intact material above the former slide may well occur.

Fig. 1 portrays a homogenous material with the existing groundwater table above the present river level, but below the final reservoir level. It is important to note that raising the reservoir level initially increases the factor of safety until the reservoir level is a little above the height of the existing groundwater. Any further rise in the reservoir level decreases the factor of safety until at some level of partial submergence (shown in Fig. 1 at el 567) it reaches its lowest value. From this point the factor of safety increases until at total submergence its value is similar to that prior to flooding. The foregoing assumes a rate of flooding which permits piezometric adjustments to occur throughout the slope. If flooding is too rapid for this to occur then higher safety factors temporarily result which progressively reduce as water permeates into the slope. Rapid drawdown after filling can decrease stability. Although Fig. 1 shows the case of a block slide along a pre-existing failure surface, colluvial, slumped or slide debris material overlying intact material are similar situations which would also fail slowly.
Fig. 1. An example of the effect of reservoir filling on slope stability.

Examples of reduction in slope stability due to flooding and resulting landslides are commonly reported, but documented cases of increased stability due to flooding of the upper part of a slide are not so common. The recorded movements in Fig. 2 were taken from a small recently flooded reservoir in southwest British Columbia. Slow movements of a pre-existing overburden slide developed on flooding and as a consequence the reservoir level was held short of the full supply level. It was found however that the movements could be slowed and in fact halted by allowing the reservoir level to increase.

The mechanism illustrated in Fig. 1 also applies to slopes which are close to sliding, although they may show little outward sign of being so. A reduction in shearing resistance at the toe of the slope along a plane or weakness which may already exist (e.g. a shear plane) or is in the process of progressive failure, may be sufficient to cause a slide. Rapid slides in sensitive clays and loose silt are to be expected. It is also generally assumed that slides developing in intact or near intact rocks fail rapidly. However a recent extensive literature search (Hungr) has indicated that such an assumption may only be applicable to hard rock or sequences containing massive sedimentary rocks, and that no cases have been reported of rapid landslides originating from thickly bedded flat lying shales.

**Beaching**

Progressive beaching of oversteep slopes within the drawdown range of the reservoir can result in considerable bank regression. Beaching also often involves small scale sliding and sloughing.

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As beaching occurs, material will be removed from a steep slope within the zone of influence of the waves and redeposited below the zone. In the case of a reservoir with a fluctuating surface level (and for that matter a natural lake) the location of the wave influence zone will vary throughout the year. This concept is illustrated in Fig. 3. It is apparent from this figure that the extent of shoreline regression at any given time will depend upon:

- the extent and permanence of the redeposited material.
- the beach slope and its stage of development.

The former is largely dependent upon the nature of the material in the slope. If the material contains a large quantity of fines, the volume of redeposited material will be appreciably smaller than the volume of material removed by wave action. The fine materials will tend to go into suspension and be carried well away from the slope. For the purpose of predicting shoreline regression it is normally assumed that silt sizes will not contribute to the redeposited material, whereas gravel and cobbles will do so. Littoral drift and possible reversions on drawdown to river conditions thus permitting erosion and removal of the redeposited material must also be considered.

The author over several years has observed and gathered information on long term beach slopes around natural lakes and older reservoirs. Some typical values are given in Table 1. It is known that long term beach slopes (i.e. those which have developed over thousands of years in lakes subject to natural drawdown) are not normally reached during the economic life of a hydroelectric project except in the upper range of the drawdown zone. This depends on the operation of the plant, but typically a storage reservoir spends a much longer period of time at or near the full pool level than it does at intermediate levels. Furthermore, material eroded from the upper levels of the drawdown range "heals" erosion at lower levels. Thus the long term beach slope extending over the entire drawdown range as shown in Fig. 3 is simplistic and may be excessively conservative. In practice a composite slope is often selected which takes into account all the considerations outlined above specific to the site.

Table 1 - Typical long term beach slopes

<table>
<thead>
<tr>
<th>MATERIAL*</th>
<th>TYPICAL BEACH SLOPE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Limited Exposure **</td>
</tr>
<tr>
<td>Silt, fine sand</td>
<td>1:8</td>
</tr>
<tr>
<td>Uniform medium sand</td>
<td>1:9</td>
</tr>
<tr>
<td>Uniform gravel</td>
<td>1:6</td>
</tr>
<tr>
<td>Cobble</td>
<td>1:5</td>
</tr>
</tbody>
</table>

*Unified Soil Classification System
**Corresponds to an effective fetch of 2 and 8 km (Davis et al.)

In summary, a natural shoreline formed by cobbles and boulders will generally erode very slowly and may show little change during the life of a project, unless the shoreline is very steep. Slopes in medium sands and gravels can be expected to erode more rapidly, but again even in the more exposed areas of the reservoir the long term beach slope is not likely to be attained at intermediate water levels. Silts and fine sands will erode most rapidly and it can normally be assumed that long term beach slopes will be reached throughout the drawdown range during the life of the project. Shoreline regression exceeding
500 m in 15 years has been measured in fine sands on a large reservoir in northern British Columbia.

SAFELINES

Definition and Purpose

A "safeline" is a conservatively located line around a proposed or existing reservoir, beyond which the security of residents and residential improvements can be reasonably assured. It is not to be confused with the predicted extent of shoreline regression (called the "breakline") which does not allow for any margin of safety.

The above terminology was established several years ago and is now in common usage on most reservoirs in British Columbia. The safeline has proven to be a very useful tool for land use control in any area that is subject to physical hazards such as landslides or periodic flooding. It may also be used as a guide to obtain easements or purchase land around new reservoir areas that are subject to sliding and beaching.

Safelines are primarily concerned with residential use of land. Thus any other actual or intended land use is not taken into account in locating the safelines. Roads and farming commonly continue within the safeline provided there is no permanent habitation. It is important to emphasize that failure and erosion of the entire land area falling below the safeline, or for that matter the breakline, would not occur. Occasionally parts of the area are expected to fail. Usually it is not practicable to predict where these failures would occur, and hence the entire suspect shoreline area is placed within the safeline.

Also a safeline represents the setback that would be required should a slide of a given size classification occur, thus the probability of occurrence of such a slide is not taken into account in establishing its location.

Around large and sparsely inhabited reservoir areas the safeline is based on limited geological and groundwater data; usually that which can be gathered from surface surveys. In areas of particular interest or more developed areas some subsurface drilling is done to supplement the surface surveys. Detailed drilling is normally only carried out where the value of the land falling within the safeline approaches or is greater than the cost of protecting (stabilizing) the shoreline, and thus the cost of refining the safeline location is warranted.

Because the location of a safeline is by definition chosen conservatively, consideration can be given after a suitable period of operation to adjusting its location towards the reservoir. Typically in a run-of-the-river plant where there is little seasonal drawdown, experience shows that any low bank property that does not develop evidence of instability during the initial five years of reservoir operation can in general be considered to remain stable throughout the life of the reservoir. Thus with this type of operation, the low bank safeline is commonly adjusted following the five year period and previously purchased property may then be resold or leased back to residential use.

Location

The distance between the safeline location and the breakline represents the margin of safety. The selection of the margin of safety depends on:

- The cause of shoreline regression. Frequently no additional setback from the breakline is used when regression is by beaching erosion only. The reason for this is twofold. First, as a result of the Columbia River Development a considerable amount of reliable information on beaching erosion has been obtained (paper in preparation). Second, regression caused by erosion normally occurs progressively and obviously, and an error on the unconservative side would not constitute a threat to the security of residents.

- The extent of the field investigation of the area (some properties are so heavily vegetated as to inhibit detailed surface inspection; other properties have been drilled, and the
EXAMPLE

AFTER FLOODING

B** C1

BEFORE FLOODING

B C1

PROBABILITY OF OCCURRENCE

(Designated using asterisks)

Two asterisks(**) — high probability of occurrence
One asterisk (*) — not imminent, but assumed to occur at sometime during the life of the development.
No asterisk — possible, but low probability

SLIDE VELOCITY

Rapid — greater than 3m/sec.
Moderate — 0.3 to 3m/sec.
Slow — 1m/year to 0.3m/sec.
Creep — less than 0.1m/year

SLIDE TYPE

Designation

A — Segments of shoreline where regression of the shoreline due to erosion and/or sloughing will be slight; also covers slopes where slides may occur but are not expected to affect the slope above full pool.

B — Segments of shoreline where small slides on terrace slopes (normally not higher than 50m) or minor slumping of higher slopes may occur (involving not more than 100,000 m$^3$ in one slide). Also includes reactivation of slide debris at the toe of higher slopes, provided the above volume limit is not exceeded.

C — Segments of shoreline where large slides may occur involving failure through bedrock either totally or in part. Such slides could range in size from 100,000 m$^3$ to several tens of millions m$^3$. The slides could affect any portion of a high slope and are not restricted to toe failures. Type C slides are subdivided into Type C1 signifying a capability of up to moderate movement, and C2 signifying a rapid movement capability.

D — Segments of shoreline where long slides may occur involving overburden only or slide debris (reactivation). Otherwise, the Type C definition applies.

Figure 4 — Typical stability classification of reservoir shoreline.

Soil conditions are well established.

The degree of uncertainty or lack of understanding of mechanism regarding future slope stability.

Typically for low bank shorelines which are subject to sliding the safe line setback from the top of the existing slope is 1.5 to 2 times the breakline setback.

CLASSIFICATION OF THE SHORELINE

Safeline studies, being concerned with the security of residents, are based upon assumed 'worst case' conditions. Planners and engineers also need to be informed about the risk or probability of occurrence landslides around the reservoir shoreline. In response to this requirement a shoreline classification system has been established as depicted in Fig. 4.

The system recognizes that a segment of shoreline may be subject to more than one type of slide potential each with a different probability of occurrence. An example of a classification of a large slope greater than 50 m high consisting of overburden overlying bedrock is given below:

B** C1 D2*

Within this shoreline segment reactivation of up to 100,000 m$^3$ of slide debris is expected; it should also be assumed that a large rapidly moving slide of the overburden (involving more than 100,000 m$^3$) will occur within the life of the development; there is also a possible but low probability of a large slowly moving slide involving the underlying bedrock.

The system also readily permits comparisons of slope conditions before flooding with predicted conditions after flooding, thus showing the effect of flooding. A shoreline segment is given two classifications within a circle (Fig. 4). The upper half represents the after flooding condition and the lower half the pre-flooding condition.
It should be noted that the terminology used to describe slide velocity differs from that previously suggested by others (Varnes). The term 'rapid' is only applied to those slides that have the potential for moving at greater than 5 m/sec. Model tests indicate that this is the threshold velocity required to generate significant waves, i.e. more than 0.5 m high (Slingerland and Voight).

The use of the classification with regard to probability of occurrence is expanded below:

- High probability (**) indicates that a slide can occur at any time. It is usually applied to those slopes which are known to be presently active.

- Moderately high probability (*) is applied to those areas where a slide or slides of a designated classification should, for the purpose of studies concerning wave hazards, land use and safelines, be assumed to occur during the life of the development. Overburden slopes showing no current signs of instability but which are appreciably steeper than long term slopes are usually put into this category. Bedrock slopes where there is reasonable evidence that their stability is uncertain or could become uncertain are also placed in this category.

- Low probability (no asterisk) indicates a possible potential. Except for residential safeline studies it may be assumed that slide(s) would not occur within the life of the development. Overburden and bedrock slopes which show evidence of having moved in the past and which have a lengthy history of stability are placed in this category. Overburden slopes steeper than long term slopes but not appreciably so are also placed in this category.

**SLIDE GENERATED WAVE HAZARDS**

There is a substantial bank of literature covering case histories and assessments of post failure velocities based on field evidence (Hung). However our current analytical techniques for quantitatively assessing the dynamics of slides after failure are somewhat rudimentary.

Application of the principle of conservation of energy in the conventional approach to computing landslide velocities. All methods involve consideration of changing energy relationships as the slide mass follows its failure path. Potential energy less energy consumed in overcoming friction is converted into kinetic energy. However all methods suffer from lack of a reliable way to compute energy losses which in addition to frictional losses along the failure plane must include losses from internal deformation and other sources. Also it is difficult to quantitatively allow for changes in pore pressure as the slide moves extensively and thus the effect on frictional resistance. As a result the application is necessarily conservative and sometimes risks being excessively so and thus not helpful.

One approach which has been applied by B.C. Hydro and Power Authority in British Columbia is illustrated in Fig 5. It recognizes that with curvilinear failure the slide mass deforms. Briefly the method consists of estimating five or so profiles spanning the pre-failure to the failed conditions. The initial profile is divided into blocks and an attempt is made to identify each block of material and its centre of gravity in the subsequent profile. By summing the potential energy changes and frictional energy losses for each block, the kinetic energy of the slide mass and thus the velocity of the toe block can be computed for each profile. This permits a plot of slide velocity versus distance travelled to be compiled (Fig 5). The assessment of frictional energy losses is facilitated by back analyses of existing slide masses in the area. The hydraulic model tests are carried out using a selected slide profile and a computerized mechanism is used to duplicate the velocity plot.
Typically a 4-year schedule is now required to carry out shoreline stability assessments for large hydroelectric projects in Western Canada. The model program outlined below is idealized, and in practice some variation from it is frequently the rule.

**Year One:** Preliminary shoreline stability assessment to establish the nature, magnitude and location of any problems. The shoreline may be classified with respect to existing and future stability, but the probability of occurrence of slides is not considered. For large projects, this work is carried out using 1:250,000 mapping. Our practice is to use existing information and a detailed air photo interpretation plus a brief field check. Several project development schemes are frequently considered.

**Year Two:** Phase I of the detailed shoreline stability assessment normally follows selection of the preferred development scheme. It covers completion of all pertinent geological mapping to a scale of 1:50,000. Detailed surface geological surveys (at 1:5,000 scale) of selected areas is carried out to improve understanding of the mechanics of potential slide areas and, within broad limits, quantities of potential slide masses. An attempt is made in this phase to classify the slide areas in terms of probability of occurrence. The residential safe line is established using 1:50,000 scale maps. The advice of hydraulic engineers concerned with wave hazards is often sought in this phase to assist in selecting the areas to be geologically mapped. This phase does not normally include subsurface exploration by drilling, monitoring of movements, or hydraulic model tests, but leads to recommendations regarding such programmes.

**Years Three and Four:** Phase II of the detailed shoreline stability assessment covers all necessary subsurface drilling, sampling and instrumentation of slide areas with problem potential. The laboratory testing of samples is carried out simultaneously with the drilling programme but frequently lags behind it by several months. Movements of selected slides under existing conditions are monitored. The end result is a compilation of data including piezo-

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**Assessment of wave hazard potential**

Assessment of wave hazard potential should be a cooperative exercise involving the geologist and the hydraulic engineer. The geologist frequently finds himself in the unenviable position of having to establish the worst reasonable case to be subsequently tested by the hydraulic engineer. A better and less time consuming approach is for the geologist to first complete sufficient surface surveys to identify and define potential slide areas and then for the hydraulic engineer to determine the limits of slide magnitude and velocity that would create problem waves at a specific site. Based on this information, the geologist can usually eliminate several slide areas from further consideration and concentrate on those remaining.

**Program of Investigation**

The question of allowing sufficient lead time to permit adequate field investigations and studies is always a key item in any large project.
metric pressures, slope movements, failure plane locations, shear strength parameters, etc. The latter part of this phase consists of the selection of an appropriate method of slope stability analysis, performing the analysis, and interpreting the results.

Where potential for wave hazards cannot be ruled out, Phase II analyses are carried out simultaneously with hydraulic model tests.

From time to time throughout this phase of the work detailed safineline locations are provided on sections of shoreline of particular interest. The scale of mapping required for this work is 1:5,000 or better.

Post-Filling Years: Phase III of the shoreline stability assessment consists of monitoring of the performance of slopes under reservoir operating conditions. For run-of-the-river plants or those reservoirs that are consistently drawn down annually over the same elevation range, revisions can be made to the safineline (relaxing or removing it) after a period of about 5 years.

CONCLUDING REMARKS

This paper is primarily directed toward those responsible for planning and administration of reservoir shoreline studies for hydroelectric and other water resource developments.

The need for a reliable assessment of the shoreline stability of reservoirs arises out of a desire that there be no surprises. The security of the dam and shoreline developments must be assured; and the interests of shoreline residents and other users of the reservoir area must be protected. The approach described in this paper, which has been developed over a period of several years in Western Canada, satisfies this requirement.

One of the most important outcomes of such a comprehensive program is that it forces consideration of the whole reservoir area and raises the level of consciousness of reservoir shoreline problems.

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ON THE INTERACTION BETWEEN THE ENGINEERING CONSTRUCTION AND GEOLOGIC ENVIRONMENT IN THE MAN-MADE LAKE AREA

L'ACTION RECIPROQUE ENTRE LA CONSTRUCTION D'INGENIEUR ET L'ENVIRONNEMENT GEOLOGIQUE DANS LES REGIONS DES LACS ARTIFICIELS

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ABSTRACT

This paper deals with the discussion of the interaction between the engineering construction and the geologic environment in the man-made lake area. The engineering construction causes the changes of geologic environment which can inversely affect the engineering buildings in the area. The case-histories of reservoir induced earthquakes in the Xingfenjiang reservoir area and reservoir landslide Tangyankuang in the area of Zhaxi reservoir indicate the important role of underground water on the change of the environment and on the instability of earth's crust and reservoir slopes. Therefore, the sensitivity of hydrogeologic structure with regard to reservoir water is an indication for the engineering geological evaluation. The careful regulation of water level in reservoir will be an effective measure to control the engineering geomechanical processes in the reservoir area.

Cet article traite de l'action reciproque entre la construction d'ingenieur et l'environnement geologique dans les regions du reservoir d'eau. La construction du reservoir provoque la variation du environnement geologique qui influence inversement aux ouvrages d'ingenieur dans cette region. Les exemples du tremblement de terre dans la region du reservoir d'eau Xingfenjiang et du glissement de roche Tangyankuang dans la region du reservoir d'eau Zhaxi indiquent au role important d'eau souterraine en la variation de l'environnement et la perte de la stabilité de la creuse terrestre et la stabilité des pentes. C'est pourquoi la sensibilite de la structure hydrogeologique a l'egard du reservoir est une indication pour la evaluation geologique de l'ingenieur. Le reglement du niveau d'eau dans le reservoir est un moyen gouvernant le processus geologique de l'ingenieur.

Introduction

The engineering geological evaluation of the area of the engineering sites is an important basis for planning and designing works. The investigation and recognition of the engineering geo-
logic environment, including natural environment and geologic conditions are the prerequisite for the engineering geologic evaluation. However, the study of the relationship and interaction between the engineering construction and the geologic environment will provide with the theoretic basis for the engineering geological evaluation. If such interaction is correctly understood and grasped, then it is possible to make a reliable evaluation and successful prediction.

This paper specially deals with the interaction between engineering construction and geologic environment in the area of man-made lake. The main factor causing the engineering geomechanical processes in the area of man-made lake is reservoir water. Prediction of potential future change in hydro-geologic conditions is an essential part of the engineering geological evaluation.

Basic Considerations

The engineering activity of mankind is closely related to the superficial surface of the earth, i.e. earth's crust. Particularly, the upper earth's crust is the most important objective for the engineering geological study. The terrain units of the upper crust of the earth are occupied by geologic formations and composed of geologic mass formed in the lasting geological history. The combination of geologic masses with certain regularity constitutes the geologic structure in a region and essentially determines the character of the engineering geologic environment.

There are two aspects reflecting the interaction between the engineering construction and the geologic environment: (1) The influence of natural geologic processes on the engineering construction, for instance, the landslides may cause the risks for dam construction; (2) The back feeding influence of the geologic environment due to its changes caused by the engineering constructions in the region. The engineering geological prediction of the latter is definitely important and more difficult. Most of major accidents in the geological engineering are due to lack of deep understanding in such back feeding process.

The engineering construction in the interaction process to be studied here belongs to the artificial factors. There are three aspects of the influencing action of man-made lake construc-

(1) Loading effect of building structures

There are a series of building structures to be constructed in connection with the reservoir, such as dam, water conduit tunnels, spillway and sluice. They can affect the geologic mass in foundation by static and dynamic loading, causing deformation and stress readjustment in geologic mass. And due to this action some changes of natural state and physico-mechanical properties, especially the filtration coefficient, in geologic process in the site.

(2) Effect of construction

The rock and slope excavation by blasting in the construction of dam, head buildings of reservoir and roads, bridges in the area exposes the fresh rock mass into surface and causes its stress releasing, loosening and weathering.

(3) Effect of water mass in the reservoir

The influence of a man-made lake on the geologic environments is mainly realized by the water mass in it. The hydrodynamic, hydrochemical processes and accompanied variation of the regime of underground water will induce various engineering geological phenomena and changes in the hydrogeologic conditions.

Under the action of above-mentioned three main factors, a series of various types of engineering geologic processes can be induced in the reservoir area, such as the deformation and stress readjustment due to loading by building structure, weathering, relaxation, loosening and vibration due to blasting and excavation; erosion, karstification, solution, collapsing and filtration causing seepage flow, pressure and softening of geologic mass, due to the effect of water mass in the reservoir.

The above-mentioned engineering geologic process induced by the change in the geologic environment can severely affect the engineering constructions in the reservoir area. The feeding back effects are as following:

(1) The engineering geologic process affecting the stability of reservoir head structures: instability and deformation of geologic mass, erosion by spilled water flow, and etc.;

(2) The engineering geologic processes affecting the buildings on the reservoir banks, such as reformation of
reservoir banks, reservoir landslides, slope collapse, and subsidence;
(3) The engineering geologic processes affecting the reservoir itself, such as siltation and solid flow;
(4) The engineering geologic processes affecting the regional stability, as earthquakes induced by water impounding in reservoir.

The above-mentioned engineering geologic processes can be of severe influence not only on the buildings having been constructed, but also on the engineering geologic conditions for the future constructions (Fig. 7)

Crustal Stability under the Action of Reservoir Impounding
Up to present time, more than hundred main shocks have taken place due to reservoir impounding over the whole world. The time of shocks, distribution of epicenters and seismic activity are closely correlated with water-lifting and descending in the concerned reservoir. This phenomena can be considered as crustal instability caused by engineering activity of mankind.

Although some different concepts exist in recognition of the mechanism of reservoir induced earthquake, but all of them do not exclude the role of reservoir water, for instance, loading

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effect, water temperature effect, effect of infiltration pressure, and etc. Therefore, for the first insight, the reservoir induced seismicity can be concerned as result of changes of geologic environment in terms of tectonic stress adjustment under the action of water impounding in the reservoir.

The earthquakes in the Xinfengjiang Reservoir Region can serve a typical example of reservoir induced earthquakes. It is a single buttress dam of height 105 m. After impounding the reservoir a series of minor shocks was observed and its frequency gradually increased. According to this situation the strengthening measures were taken immediately for dam protection. Just after the protective works of first stage having been completed the main shock of 6.1 magnitude took place on March 19, 1962. Since then, more than two hundred thousand small shocks were recorded. Because of the dam strengthening by filling concrete into void space between buttresses the capacity of the dam resisting the lateral vibration was much increased and the collapse of the dam then was avoided, with exception of an open crack of length 82 m occurred on the top of left dam shoulder.

The study of this reservoir induced earthquake shows that the change of geologic environment due to reservoir water is the main cause of the induced seismicity. In view of that the water loading and temperature effect are limited on the surface parts of the reservoir bottom, recently most of researchers emphasis the effect of seepage pressure. Some authors concern the increment of water head in reservoir as the trigger pressure, but the whole water pressure from surface to the focus of earthquake seems to be the trigger factor.

The seepage pressure is considered to be formed due to penetration of underground water and propagation of fractures into depth. Then the formed whole head of seepage water pressure causes the change of stress state and decrease of strength of fault material, and eventually happens instability. The study of the geomechanics background of the Xinfengjiang Reservoir region and the developing process of earthquakes gives positive support to the conclusion that the reservoir induced earthquake is caused due to the changes in the geologic environment, especially the changes of hydrogeologic conditions.

![Fig.2 Regional Geologic Background of Xinfengjiang Reservoir](image)

1. Granite; 2. Red sandstone shale; 3. Faults; 4. Depth of watertight Rock

Fig.2 shows the geologic structure in the region of Xinfengjiang Reservoir. The bottom of Xinfengjiang Reservoir is composed of granite massive forming a higher topography. The area surrounding the reservoir are covered by mesozoic red deposits. Down the stream there is a red basic with large thickness of tertiary deposits and quaternary basaltic flow. The regional He yuan Fault of polycyclical tectonic activity is going through the dam area and constitutes the boundary of the tertiary basin. Some thermal springs appear along the fault line. The abovementioned geologic background indicates that the area of Xinfengjiang Reservoir is located in the boundary between uplifting and subsident zones and the situation is favorable for penetration and propagation of seepage water into depth. The earth's crust in this region is relatively unstable, The characteristics of the developing process of Xinfengjiang reservoir induced earthquakes are longterm foreshocks and aftershocks (Fig.3). The average depth of focus of earlier earthquakes is about 2-3 Km, which may represent the initial depth of circulation of underground water. Therefore, the tectonic and gravity stresses and seepage pressure are in equilibrium at that depth and the penetration of water into greater depth is restrained.

According to the earthquake mechanism solution of the main shock (Fig.4), the active tectonic pressure is in NW-SEE direction, the earthquake fracture is striking in NW-SE direction. The
The equilibrium condition can be expressed by

$$
\sigma_{nt} - \frac{\gamma}{2} \gamma' H - \sigma_w = \sigma_s
$$

$\sigma_w$ - seepage pressure in joints at depth $H$,
$\gamma'$ - seepic weight of rocks.

For the Xingfengjiang Reservoir the lower limit of earthquakes is about 10 Km, then we have

$$
P_t = (\sigma_0 + \sigma_w - \frac{\gamma}{2} \gamma' H) \sqrt{\frac{\gamma^2 + tg^2 \varphi}{\gamma^2 (1+tg^2 \varphi)}}
$$

where $\gamma = 0.3, \alpha = 45^0$

Therefore, the focus stresses before main shock at depth 5 Km, are as follows, (Fig. 5).

$$
\sigma_1 = P_t + \frac{\gamma}{2} \gamma' H = 3220 \text{ bars}
$$

$$
\sigma_2 = \frac{\gamma}{2} P_t + \frac{\gamma}{2} \gamma' H = 1260 \text{ bars}
$$

Fig. 4. Mechanism of Reservoir Induced Earthquake

- $P_t$ - Seepage pressure
- $\sigma_1, \sigma_2$ - Principal stresses

The magnitude of active tectonic pressure can be determined taking into account the equilibrium condition of restraining the penetration of seepage water into depth by a critical effective pressure for closing the fracture, and stopping the water penetration ($\sigma_0 = 1000$ bars).

Assuming an active tectonic pressure $P_t$ acts at depth where the seepage filtration is essentially restrained, and the angle between $P_t$ and the earthquake fault is $\alpha$, then we have the normal stress $\sigma_{nt}$ on the fracture due to tectonic pressure $P_t$

$$
\sigma_{nt} = P_t \sqrt{\frac{\gamma^2 (1+tg^2 \alpha)}{\gamma^2 + tg^2 \alpha}}
$$

$$
\xi = \frac{\gamma'}{1-\gamma'} \text{ - coefficient of lateral pressure}
$$

Fig. 5. Crustal Stress State at Focus of Main Shock of Earthquakes

1. Before impounding of reservoir;
2. Main shock; 3. Dry; 4. Saturated

The focus stresses during main shock happens due to seepage pressure are

$$
\sigma_1 = \sigma_1 - \sigma_w = 2720 \text{ bars}
$$

$$
\sigma_2 = \sigma_2 - \sigma_w = 760 \text{ bars}
$$

The obtained stresses are consistent with failure concept of sliding along preexisting fractures. The factor of stability against sliding can be expressed by principal stresses.

$$
\gamma = \frac{\sigma_1 (\frac{1}{2} (\sigma_1 + \sigma_2) - \frac{1}{4} (\sigma_1 - \sigma_2) \cos 2\alpha) tg^2 \varphi}{\frac{1}{2} (\sigma_1 - \sigma_2) sin 2\alpha}
$$

where $\alpha$ - angle between normal of the fracture and $\sigma_1$; $tg \varphi, S_1$ - shear strength parameters of the fracture.

In the case of Xingfengjiang Reservoir according in situ shear tests we have

$$
tg \varphi = 0.585, S_1 = 0.42 \text{ bar(dry)}
$$

VII.61
\[ ty = 0.495, \quad S_1 = 0.2 \text{ bar (saturated)} \]

Therefore, we obtain the factors of stability at earthquake source before reservoir impounding \( \gamma_1 \), after impounding without rock softening \( \gamma_2 \), and after impounding with rock softening by seepage water \( \gamma_3 \):

\[
\gamma_1 = 1.3371 \\
\gamma_2 = 1.0387 \\
\gamma_3 = 0.8789
\]

The figures show that the reservoir induced earthquakes are caused by the combinative action of seepage pressure and softening of fracture material due to seepage water. Summarizing the above-mentioned facts, we come to a conclusion that the occurrence of reservoir induced earthquake presents the process of changes of hydrogeologic environment and seepage penetration of water into depth due to reservoir impounding. The main points of such process are as follows: (1) In the man-made lake area exist conditions of deep circulation of underground water. (2) The depth of circulation of water depends upon the equilibrium between geostress and seepage pressure. (3) The deformation of reservoir bottom due to water loading and temperature effect can promote the penetration of water into depth. (4) The water pressure and changes of infiltration conditions lead to penetration and propagation of water into great depth. (5) The increase of seepage pressure and strength softening form the earthquake source and lead to breaking of earthquakes. The minor shocks lead to main shock forming an earthquake series.

Slope Stability under the Action of Reservoir Water

It is similar to the reservoir induced earthquakes, the main cause of reservoir induced slope instability is the changes of hydrogeologic environment.

The reservoir induced landslides present an important problem for man-made lake construction. The large Vajont valley landslides caused severe hazards and damage in the downstream area. The Tangyankuang rock slide is a typical reservoir induced instability. The study of this rock slide enables us to understand the important role of water impounding in the change of hydrogeologic environment.

The Tangyankuang rock slide is located in the area of Zhaxi Reservoir, 7.5 km from the dam. The slope is composed of sandstone interbedded with slate. The layer is dipping into reservoir. Some weak intercalations exist among the slate layers. The rock slide happened during the beginning of water impounding. The volume of the rock slide is about 1.65 million m³ and the wave reaches 21 m height. The geologic boundaries are weak intercalation as sliding surface, and faults F₁, F₂ as lateral separating surfaces (Fig.6). Therefore, before water impounding of the reservoir the geologic boundaries have already existed, and the slope is in a potentially unstable state. After impounding of the reservoir, the level of underground water is lifted, softening the intercalations. In addition, there was heavy rain lasting 8 days with amount of rain fall about 1200 mm. Under this situation the slide happened.

According to the conditions of Tangyankuang reservoir rockslide a numerical simulation by means of the finite element method is conducted for researching the influencing factors of the rockslide. Fig.7a shows the slope section for computation. During water level lifting to the sliding mass, the displacement increases and the factor of stability against sliding decreases. Before water impounding the factor of stability reaches 1.7 and it drops to 1.2 due to hydrostatic effect of lifting of underground water after water impounding in reservoir. Taking into account the softening effect on the weak intercalations, the factor of stability probably decreases to 1.0, i.e. the critical state. (Fig. 7c).

To simulate the action of heavy rainfall the open cracks developed on the top of slope during slope deformation.
Fig. 7. Finite Element Simulation of Reservoir Landslide

I. Initial Condition; II. Uplifting of Water Level; III. Rainfall

don due to water impounding in reservoir are considered to be filled with water causing lateral pressure. In that case the computed factor of stability becomes lower than 1.0. The above-mentioned values of factor of slope stability can continue decreasing due to drawing off water from reservoir, while underground water keeps its high level forming large hydrodynamic gradient and hydro-static head between underground water and reservoir water. In this case the slope is easily to slide downwards to reservoir. Fig. 7b shows the distribution of local factor of stability along sliding surface. Fig. 8 shows the stress distribution along sliding surface. Summarizing the above-mentioned points, the change in the regime of underground water is considered to be the most important factor inducing the landslides along the reservoir banks.

Practical conclusions

For the engineering constructions in the man-made lake area the feeding back influence due to the action of reservoir on the geologic environment should be taken into account. The owner of reservoir usually is interested in knowing the influence of reservoir on the related hydraulic works. However, the influence on other and future civil engineering works in the reservoir area is of great significance and it is an important task of the environmental study of engineering geology.

The engineering construction of man-made lake has a series of effects on the geologic environment, among them the water mass is the most powerful factor causing the change in hydrogeological regime and conditions. The latter might induce the crustal and slope instabilities and lead to the engineering geologic hazards.

The action of reservoir on the geologic environment is controlled by the geologic structure in the area. The hydro-geologic structure which is favorable for deep circulation and a certain level of tectonic stresses are the basic conditions for occurrence of reservoir induced earthquakes. Similarly, the existence of unstable rockmass structure is the prerequisite of triggering a reservoir rock slide. Therefore, when the geologic boundary surfaces are in intersection forming the
unstable blocks on the slope with a factor of stability against sliding less than 2.0, the slope safety should be cautiously concerned. The characteristics hydro-geologic structure and sensitivity of geologic structure to action of water mass should be an important indication for the engineering geological evaluation of reservoir construction. In view of the dependence of reservoir induced earthquakes and rockslides upon the seepage effects the rational regulation of water level in reservoir especially in the early stage of water impounding will be one of effective measures to control the reservoir induced seismicity and instability of rock slopes.

References

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DEFORMATIONS OF RESERVOIR SHORES IN LOESS REGIONS

DEFORMATIONS DES COTES DE RESERVOIR AUX REGIONS LOESS

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Xi'an, Shaanxi
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ABSTRACT

Making use of the data collected through many years of observation on the behaviour of existing reservoirs, a discussion on the characteristics of deformation of reservoir shores in loessic regions, the controlling factors thereof and methods of prediction is presented. Generally, deformations that take place along reservoir shores are of numerous types, such as toppling of slopes, slides, slumps, stripping of slope surfaces and the erosion-abrasion process, resulting in the formation of relatively stable shore slopes which in their turn can be classified into three categories, namely abrasional, accumulative, and abrasion-accumulative. These deformations evolve rapidly, usually taking up only a short duration of time in each individual process. As a result, the volume of slope materials involved in deformations taking place in a period of two to three years immediately after the initiation of storage of water can reach up to as high as 60-80% of the total possible amount predicted for the whole lifetime of the reservoir. After that, the evolution of deformations slows down and finally ceases.

Here, the factors that affect the deformation processes most are the lithology of rocks and soils, structural and morphological characteristics of shore slopes, wave action and deposition. Presently, the method of engineering geological analogy is recommended for use in predicting possible behaviour of existing shore slopes.

ABSTRACT

Utilisant les données obtenus à travers les plusieurs années d'observation sur le comportement des réservoirs existants, une discussion sur les caractéristiques de déformation des rives de réservoirs dans les régions loessiques, les facteurs qui les contrôlent et les méthodes de prévision à cet égard est présentée. En général, les déformations qui ont lieu le long des rives de réservoirs sont nombreux en types et on peut compter des éboulements de pentes, glissements, affaissements de terrain, enlèvements de couches superficielles et le processus d'érosion-abrasion. Par conséquent, en résulte la formation des rives-pentes relativement stables que l'on peut classifie en trois catégories: abrasionelle, accumulationelle et abasion-accumulationelle.

Les déformations évoluent rapidement, ayant besoin d'une durée très courte de temps pour l'évolution complète de chacun processus individuel. Le volume des matériaux impliqués dans les déformations qui ont lieu dans une période de 2-3 ans directement après le remplissage du réservoir d'eau peut atteindre une proportion qui est de 60-80% du volume total prévu pour toute la durée d'existence du réservoir. Après cela l'évolution de déformations se ralentit et tend à la cessation. Ici, les facteurs les plus influentiels sont les suivants, la lithologie des roches et sols, les caractéristiques structurales et morphologiques des rives-pentes, l'action des vagues et la sédimentation.
Pour le présent, on recommande la méthode de l'analogie de la géologie de l'ingénieur pour la prédiction du comportement de rives-pentes existantes.

Loess or loessic soils, as they are generally called, actually fall into two major groups, namely loess or loessic soils and loesslike soils, the latter being further divided into loesslike lean clays, loesslike sandy clays and silty clays depending on their granulometric composition. Based on the time of deposition, all the above-mentioned soils may be distinguished as ancient loess (or paleo-loess), new loess (or neoloess) and recent loess. All ancient loessic soils are invariable loess-like soils. The term’s ‘loess regions’ is used for those regions where the predominant strata are loessic soils or loessic soils with intercalated sands and gravels, underlain by other strata.

1. Characteristics of deformation of reservoir shores in loess regions.

1.1. Deformations of reservoir shores begin to appear and evolve immediately after impounding, taking the forms of topplings (collapses) of slopes, slides, slumps, stripping of surficial soils and erosion-abrasional processes of which topplings of slopes are the most frequent and consequently most significant in terms of the influence they exercise on reservoirs. "Collapse of slopes" is a term used by many people as a general designation of all these forms of slope deformations.

1.2. Depending on the structural and lithological features, hydrographical characteristics, mode of operation and time duration of sustained water levels, which differ for different reservoir sites and their shore slopes, various relatively stable shore slopes are finally formed as a result of evolution. They can be classified into three categories, namely abrasional, accumulative and abrasion-accumulative shore slopes, as shown in Table 1.

1.3. Deformations of shore slopes begin to appear and evolve right after the level of stored water has for the first time exceeded the local flood level of the original river-basin. The process is characterized by a high rapidity and a short duration, begin most intense during the initial period of storage (impounding). Take Guantung Reservoir as an example.

Storage of water there started in August 1955 and most of the shore slopes reached a new state of stability and ceased to evolve around 1960. However, out of the total volume of slope materials involved in deformations, 50-80% was due to the deformations taking place in the period of 1955-1956. Similarly, in the loess region of Shaanxi Province the width of shore slopes involved in deformation processes during the first year after impounding, ranges from some meters to over ten meters for large and medium size reservoirs. After two to three years of storage, the amount of slope materials that have taken part in deformations may reach up to 70-80% of the final amount, taking up 1-3.7% of the total storage volume or standing for 4.5-13.7% of the total sedimentary volume. The deformation process slows down with the lapse of time and finally ceases. For instance, storage of water in Lingho Reservoir of Shaanxi Province was initiated in 1980. Three to four years later, deformation processes of shore slopes came to an end. For this reservoir, the width of shore slopes involved in the processes was 1-15 m and the angle of repose (\(\alpha\)) of shallow beaches were 24°-37° (Fig. 1). Naturally, deformations differ with reservoirs of different capacities and likewise with different parts of one and the same reservoir. They are less serious in gorge-type reservoirs than in lake-type ones, while for one and the same reservoir they are more intense in downstream and middle-reach districts than in its upstream parts.

Fig. 1.

2. Factors affecting deformations of shore slopes in loess regions

Numerous factors exercise their influence on shore slope deformations, but what affect these processes most are the lithology of rocks and soils, morphological and structural features of shore slopes, wave actions and sedimentation.

2.1. Lithology of local rocks and soils.

As is well known, the physical, mecha-
Table 1. Main categories of relatively stable shore slopes formed in reservoirs

<table>
<thead>
<tr>
<th>Category</th>
<th>Schematic profile</th>
<th>Characteristics and distribution of shore slopes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abrasional shore slopes</td>
<td></td>
<td>No colluvial deposits are found above the dead water level. However, the colluvial materials keep on intermingling with reservoir sediments and accumulating in flat lying layer with the lapse of time until the slopes are covered up and the life of the reservoir is brought to an end.</td>
</tr>
<tr>
<td>Quintuple-turn slopes</td>
<td><img src="image" alt="Quintuple-turn slopes" /></td>
<td>Shore slopes of this kind are found: (1) in reservoirs for everyear storage with large storage capacities, where the design water level is rarely reached; (2) in the district near the dam of reservoirs where the water surface is broad and the water is deep; and (3) during the initial operation period of reservoirs.</td>
</tr>
<tr>
<td>Accumulative shore slopes</td>
<td></td>
<td>Shore slopes of this kind are widely distributed, the materials reaching usually up to the design water level but seldom the high water level.</td>
</tr>
<tr>
<td>Quadruple-turn slopes</td>
<td><img src="image" alt="Quadruple-turn slopes" /></td>
<td>Shore slopes of this kind are found: (1) in reservoirs where the rate of sedimentation is high, (2) in upstream districts of reservoir; and (3) during the later operation stages of reservoirs.</td>
</tr>
<tr>
<td>Triple-turn slopes</td>
<td></td>
<td>Shore slopes of this kind are found: (1) in small reservoirs with rapid fluctuations of the water level; and (2) in influent channels within reservoirs as well as upstream districts of reservoirs where wave actions are slight.</td>
</tr>
<tr>
<td>Double-turn slopes</td>
<td><img src="image" alt="Double-turn slopes" /></td>
<td>The distribution of this category is relatively wide and common. Shore slopes of this category are intermediate between the other two categories, the colluvial materials being not abundant enough to cover up entire slopes.</td>
</tr>
<tr>
<td>Abrasion-accumulative shore slopes</td>
<td><img src="image" alt="Abrasion-accumulative shore slopes" /></td>
<td></td>
</tr>
</tbody>
</table>

**Note:** The images of the schematic profiles are not included in this description.
rical as well as hygroscopical properties of loessic soils differ with their time of deposition, genetical conditions, soil types and their location of distribution leading to corresponding disparities concerning the mode and extent of deformation. Take the stable angle of shore slopes as an example. Its value varies with soils differing in time of deposition in the following order:

Angle of ancient loess of Lower Pleistocene ($Q_1$) that of ancient loess of Middle Pleistocene ($Q_2$) that of loess of Upper Pleistocene ($Q_3$) that of recent loess of Holocene ($Q_4$).

As shown by investigations, for reservoirs in the loess region of Shaanxi Province the angle of abrasion after two, to three year of impounding varies as follows:

Silty clays of $Q_1$ 30°-40° Loess-like lean clays of $Q_2$ 30°-35° Loess of $Q_3$ 20°-30° Recent loess of $Q_4$ less than 20° Silts and Silty sands are apt to turn into a liquid-plastic state with a resulting slope angle of 3°-5°. Furthermore, the angle of abrasion ($\alpha_2$) for regions east of the Liupanshan Range is larger than that for regions west of it.

It is held by some authors that the shallow beach angle is dependent on the granulometrical composition of the soil mass concerned and that shallow beaches consisting of clayey soils have the least angle of deposition. However, results of investigations show the contrary to be true. Based on these results in my opinion is that the stable slope angle depends on the structural strength and the later in its turn is loosely related to the amount of soluble salts as well as the clay particle content of the soil concerned. The silty clays of Lower Pleistocene ($Q_1$) have the largest angle of deposition of all the above-mentioned soils because of the lower content of soluble salts and the higher structural strength they have in comparison with the rest.

2.2. Morphological and structural features of shore slopes

Morphological features have their effects on the intensity and rapidity of deformation of shore slopes. Deformations are more extensive and more rapid with convex shores surrounded on three sides by water than with concave ones. They are intense where the local topography is broken or shores are steep, otherwise they are slight. Generally no deformation will take place where the shore slope is smaller than or equal to the shallow beach angle of abrasion.

Similarly, the mode and extent of deformation vary with the structural features of slopes. There are three types of non-homogeneous and multi-layered shore slopes in loess regions: (1) shore slopes composed of slide debris and colluvial materials; (2) multi-layered shore slopes with wrenched structural zones or planes; and (3) shore slopes of river terraces comprising highly collapsible loess and underlying sands and gravels. The former two types are apt to deform by sliding, while the latter one is characterized by tension cracks due to ground subsidence on wetting as well as slumps of terraces, often reaching a great extent.

2.3. Wave actions.

One of the effects of wave actions is the breaking up of soil masses with the temporary formation of caves and inverse slopes and resulting topplings and slides. The second effect is the pulverization and transportation of the debris. These two effects alternate in close sequence, resulting in a destruction process of continuous, yet cyclic nature for shore slopes, until a stable equilibrium is reached. The overall destructive effect of wave actions is dependent on the magnitude of wave energy, the morphological and structural features and soil types of shore slopes as well as the depth of water in front of them.

Fluctuations in water level and the time duration of various sustained water levels play a role in the reforming of shallow beaches of deposition. Shallow beaches formed under sustained high water levels usually are destroyed during following low levels. The longer the time duration of a certain sustained water level, the wider and flatter the shallow beach is formed under it. As a consequence, stepped shallow beaches are formed in the course of a general rising or lowering process of the reservoir water level with interpolated intervals of time duration during which the water level is sustained at a certain elevation (Fig. 2). Obviously, it is $\alpha_\text{av}$ not $\alpha_{\text{nor}}$ which better denotes the shallow beach slope angle.

2.4. Actions of sedimentation

Widespread denudation due to runoff exists throughout loess regions, giving rise to mud-laden river streams and serious sedimentation in reservoirs built in regions. Investigations conducted on small and medium-
size reservoirs in Shaanxi Province shows
that, up to 1970, the volume occupied by
sediments is as high as 50% of the entire
storage capacity of the province. The
situation is far more serious concerning
large reservoirs on the Yellow river. For
instance, the loss of storage capacity as
referred to the highest operation water
level incurred by Sanmenxia Reservoir
after four floods periods was close to 50%
while none of the four floods met the cri-
teria of a major flood.

Owing to the rapidity of sedimentation,
shallow beaches close to shore slopes take
up short durations of time for formation,
covering up part of the shore slopes and
enabling them to reach a new state of
stability in the process. Types of sedi-
ments and locations of their sedimentations
differ with riverstream characteristics.
The morphological features of reservoirs
and modes of operation are also different.
The center of gravity of deposition for
the shape of delta type is concentrated
chiefly on the front slope, while for the
pyramidal type it is situated on the up-
stream of the dam. The distribution for
the belt shape type of deposition is uni-
form. The transverse shape of reservoir
deposition and its distribution are closely
related to the shape of bank slope and re-
servoir operation. According to actual
surveying data the fact shows that only
40-70 per cent of incoming sediments, not
all sediments, are generally deposited in
the dead storage of sedimentation, the
rest 'are deposited in the effective sto-
rage, and especially some considerable
quantity of sediments are deposited above
the highest water level where the "tail
raising" type is performed. Thus the rate
of deformation of bank slope and the
strength are seemed to be very un-uniformly.

In summary, one can see that what
effects the deformation processes of shore
slopes most is the complex and combined
effect of numerous factors, among which
the internal factors are the influential.
It is not the factor of wave actions which
is predominant, as some people believe.
Wave action and sedimentation are two
factors acting in opposite directions. As
a consequence, their respective effects on
the intensity of shore slope deformation
are contrary to each other.

3. Prediction of deformations of shore
slopes

As we have seen above, what is essen-
tial in deformation processes of shore
slopes is the alteration brought about in
the physical and mechanical properties of
soils that comprise shore slopes and have
gone under water after impounding. This
leads to destruction of the original state
of stability or exercises a negative in-
fluence on it, causing the soil masses in-
volved to deform under new conditions
until a new state of equilibrium is attai-
ned. This process begins to evolve
immediately after impounding and never
ceases until the end of the lifetime of the
reservoir.

In view of the number of factors and
the resulting complexity involved in the
problem of prediction of shore slope de-
formations, it is considered practical to
adopt the method of engineering geological
analogy for this purpose. In so doing,
the following data must be available: (1)
Geological charts and profiles of the
reservoir area; (2) Index water levels in
the reservoir; (3) Data concerning ele-
vations reached by sedimentation in various
periods; (4) Wave action elements; and
(5) Data regarding the stable slope angles
of various districts of the reservoir.

With these data available, one pro-
ceeds to deformation prediction by way of
making graphs and plots.

In carrying out prediction, it is ad-
visable to distinguish between two types
of shore slopes, namely the homogeneous
type and the non-homogeneous type. This is done
through due consideration of the stratigraphical,
lithological, structural and morpho-
logical features of each type. Analysis of
shore slope of the homogeneous type sepa-
ately for four consecutive portions (Fig.3).

In Fig. 3 $\alpha$ denotes the stable angle
under water; $\alpha_2$ is the shallow beach angle
of abrasion; $\beta$ is the stable angle of the
portion of slope which is under the influ-
ence of changing water levels; and $\beta$ de-
notes the stable angle for that portion of
slope not affected by water. $h_w$ is the sum
of the wave climbing height and capillary
rise.
In China, in case of lack of data for analogy, the angle values shown in Table 2 for straight, homogeneous shore slopes can be used. The large values are for prediction of slope angles of five years of operation, while the smaller ones stand for final stable angles. So far as small and medium reservoirs are concerned, division of operation periods is necessary and prediction is conducted by making use of the midians in Table 2.

In view of the prominent influence of sedimentation of shore slope deformation, a study of the characteristics of sedimentation as well as the natural governing laws is necessary for prediction of the amount of sediments and rates of sedimentation in a reservoir prediction of shore slope deformation is made after that, utilizing plots of sedimentation processes and data concerning elevations reached by sediments during various time periods. Evidently, the original point (elevation) of shore slope deformation rises with the elevations attained by sedimentation.

Straight, even and homogeneous shore slopes are ripe in actual conditions. This is because loess masses are not homogeneous continuum. They differ greatly in aptitude for collapse on wetting, and various kinds of structural planes, such as bedding planes, joints, buried weathered zones of erosion and slide planes, exist in different amounts in these masses, having been formed in the course of their deposition and later evolution. The existence of respective amounts of the above-mentioned planes and likewise the actualities of the soils concerned being susceptible or not to collapse on wetting exercise in different ways their influence on the mode and extent of rupture of shore slopes. In other words, the characteristics of deformation as well as the governing factors vary with the actual conditions of each location. Consequently, any successful analysis with regard to shore slope deformation should be conducted within the framework of the actual situation of each location, whether it is a shore slope composed of highly collapsible loess, or an incomplete slope consisting of slide or toppling debris, or a non-homogeneous slope characterized by a multi-layer structure. It is usual practice to ascertain first whether or not there is a certain possibility of a major slide or toppling about to take place. Only after that, one proceeds to make predictions by layers (or portions), referring to the angles of abrasion concerned.

As to reservoirs of the gorge type,
### Table 2. Stable slopes angles for straight homogeneous slopes

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Stable angle under water ($\alpha_1$)</th>
<th>Shallow beach abrasion angle ($\alpha_2$)</th>
<th>Stable angle of zone of changing levels ($\alpha_3$)</th>
<th>Stable angle above water ($\beta$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Recent loess</td>
<td>18-20º</td>
<td>10-12-14º</td>
<td>20-22º</td>
<td>70º</td>
</tr>
<tr>
<td>Loess</td>
<td>24-28º</td>
<td>14-16-26º</td>
<td>22-26º</td>
<td>70º</td>
</tr>
<tr>
<td>Loess-like sandy clays</td>
<td>28-30º</td>
<td>26-18-22º</td>
<td>22-26º</td>
<td>65º</td>
</tr>
<tr>
<td>Loess-like lean clays</td>
<td>30-35º</td>
<td>20-22-28º</td>
<td>28-30º</td>
<td>65º</td>
</tr>
<tr>
<td>Silts and silty sands</td>
<td>33-39º</td>
<td>22-28-32º</td>
<td>30-35º</td>
<td>60º</td>
</tr>
<tr>
<td>Sands, gravels</td>
<td>25-35º</td>
<td>24-25-30º</td>
<td>27-30º</td>
<td>35-40º</td>
</tr>
</tbody>
</table>

Note: The figures are the statistical data for regions of the middle reaches of the Yellow river, the wave height being taken as one meter.

Upstream districts of large reservoirs as well as the tributary gullies, where the water surface is usually confined and narrow, the wave action is not strong and the sedimentation process is rapid. In consequence, deformation of shore slopes are not so intense. Predictions may be conducted by using abrasion angle values appropriately enlarged, except for gully mouths, protruding shores confined by water on three sides and other unusual locations.

**CONCLUSIONS**

In loess regions, intense deformation takes place during the first few years of water storage. The factors affecting the deformation processes are numerous and complex, making it imperative to conduct studies in strict conformity with the actual conditions of each case. In our opinion, the structural strength of soil masses plays a decisive role in the stability of shore slopes. For the present, it is appropriate to use the method of engineering geological analogy in prediction of shore slope deformation. As to the protection of risky areas, decisions in that regard are dependent on the size and importance of reservoirs and their influence on local economic construction and development. For large reservoirs, prediction of shore slope deformations should be made for both the end of the first five years of operation and the lifetime of the reservoir, while for medium and small reservoirs prediction may be done once and for all. All population points, industrial and mining facilities and communication links should be removed out of the bounds of risky areas where deformations within five years have been predicted. Those where deformations have been predicted for time periods later than the first five years or for the reservoir lifetime are regarded as ready and standby areas. Removal from these areas or measures for their protection may be decided on in accordance with the results of a certain time period of monitoring of the behaviour of shore slopes.

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THEORETICAL ASPECTS OF COMPLEX MODELLING FOR REGIONAL PROGNOSIS OF LANDSLIDE PROCESSES ON THE BANKS OF NATURAL AND ARTIFICIAL BASINS

ASPECTS THEORIQUES DE L'ETUDE D'ENSEMBLE SUR MODELE AUX FINS DE LA PREVISION REGIONALE DES PROCESSUS DE GLISSEMENT AUX BORDS DES LACS NATURELS ET ARTIFICIELS

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ABSTRACT

Calculations of the stability of the slopes of banks must be based on modeling their stressed state, reporting to the geodynamic field theory and electrogeodynamic analogues taking into consideration the refraction of force and isopotential field lines at the interface of different layers.

The recurrence period $T$ of probable sliding of banks or an inter-ravine block having a width $L$ in the zone of its maximum height above the erosion basis $H$ can be assessed by $L, H$ and by the $T$, known for the compared analogue, from equations whose structure includes the solar and chandler rhythms.

ABSTRAIT

Les solutions concernant la stabilité des rives des retenues d'eau doivent s'appuyer sur l'étude sur modèle de l'état de tension des pentes sur la base de la théorie du champ géodynamique et de la méthode des analogies électrogéodynamiques et tenir compte de la réfraction des lignes isopotentielles et de force du champ à l'interface de couche aux propriétés différentes.

La période $T$ du glissement probable du bloc de rives entre deux ravins, d'une largeur $L$ dans la zone de sa hauteur maximale au-dessus de la base de l'érosion $H$ peut être estimée selon $L, H$, ainsi que selon $T$ donnée pour l'analogue comparé d'après les équations dont la structure comprend les rythmes solaire et de Chandler.
The exploitation of the littoral region of great basins requires the solution of numerous engineering-geological problems. One of the most complicated of them is the problem of regional prognosing of the landslide process, its duration and the scale of development in conditions of spatially heterogeneous geologic-geomorphological milieu. The prognosing tasks for projected reservoirs are still more complicated due to the necessity of taking into account the factor of man's interference into the historically established natural surroundings. Project solutions for such areas use the typification of the natural surroundings based on regional engineering-geological maps usually containing definite information about the course of exogenous processes in time. However, the distiction of the category of the slopes' stability is qualitative, and in general, speculative. That is why in recent years the attention of the landslide problem researchers was turned towards the theoretical elaboration of methods of through complex regional prognosing of active forms of landslide process. These methods using the cybernetical modes of the similarity theory and various mathematical methods include: the method of natural analogues (L.B. Rozovsky et al.), based on $\Pi$ - theorem of the similarity theory; the method of type models of the elements of geodynamic systems (I.P. Zelinsky) practically endorsing his theoretical foundations of landslide modelling; the method of analysis of the rhythm of landslide process activating (the method of Yu.G. Balandin and V.V. Kuntzel - A.I. Sheko); the method of landslide potential (G.P. Postoyeva et al.) using the probability estimation on simple structural areas of the role of a regional set of landslide-forming factors. The method of "potential geologic risk" (USA) and "the method of prognostic mapping according to the degree of danger of natural processes" (Japan) are similar with the latter one. The factor of time in the method of natural analogues is taken into account indirectly according to classification indices [9]. Yu.G. Balandin, V.V. Kuntzel and A.I. Sheko ground arguments on heliogeophysical causes of landslide process duration. They use the possibilities of ascertaining correlative connections between the number of landslides and the characteristics of the Sun's activity. This is an arduous task because besides methodical difficulties it is connected with propensity of helioprognosing and insufficient elaboration of heliogeophysical problems of zone positions. Yu.G. Balandin develops these ideas on the basis of improving the method of natural analogues [1 - 4]. He considers the rhythms of landslide erosion of the regional plan (T) as the reflection of the combined influence
of solar activity and lunar-solar attraction upon the state of geological bodies. This evidence is more evident on the banks of large basins and reveals itself through mobile parameters of natural surroundings such as climatic and hydrological phenomena, geotectonics, geomagnetic and geoelectric fields, the change of stresses on the surface of the Earth under rotational forces variations. The solar (S = 11.18 years) and Chandler (Z = 1.242 years) rhythms which have common relations with the main member of the rotation of the Earth (18.58 years) are used in the structure of analytical operations of prognosing on the basis of these notions.

The principal bases of the method offered by the authors to estimate the regional geodynamic situation are given below.

The method of type models is methodically based on the similarity and modelling theory. The choice of the work model is preceded by engineering-geological outlining of the geological bodies. They are imagined as a solid or sufficiently solid linearly-deformed medium, subjected to the requirements of the linear theory of elasticity. The problem is considered as plain (or in section); geostatic or stationary geodynamic referring to stresses which appear under the influence of the volume force-rock weight (analogue mathematic modelling) or dislocations (physical and numerical methods) - in condition of a stable state of the slope. The examined massives may have a simple (homogeneous, quasi-homogeneous) or complex geological structure with separate-block homogeneity of rock mass. The degree of homogeneity is defined by the analysis based on the probability theory of the fields of physico-mechanical properties. For homogeneous masses the apparatus of continuous functions of mathematical analysis is used in the structure of mathematical models applied for geoschemes' descriptions. The apparatus of discontinuous and random functions is used for complex mass; the law of refraction of isopotential and force lines of geodynamic field on the strata borderline is taken into account on the basis of the principle of superposition of decisions. The choice of models is controlled from technological positions. The possibilities of methods to reveal stability fields by means of comparison between the stress fields and the strength of rocks are also taken into account. The safety margin of the object of simple (complex) structure is considered as the ratio of the mathematical expectation of the field of the rock's strength to the value of maximum (integral) tangential stress and it is defined by means of creating the field of isopotential and force lines' systems. The method allows to elucidate many practically important questions in detail [6].

The method of Yu.G. Balandin
is based on the similarity theory and on the theory of cosmo-geophysical conditionality of the development of exo-geodynamic processes in time. It uses structure system analysis of time series and methods of estimation of similarity of fields of physico-mechanic state of geological bodies.

The theory gives the opportunity to get generalized equations for concrete cases of nature parameters correlation and to pick out a group of phenomena for which a similar analytic formalisation is probable. So far in regions with similar geologic conditions [10] the parameters of the landslide process turned out to be bound by simple ratio:

\[ \frac{1}{k} T_i : T_j = L_i \overline{H}_i \overline{L}_i : (L_j \overline{H}_j \overline{L}_j) = P_i : P_j \]  
(1)

\[ L_i \overline{H}_i \overline{L}_i : (L_j \overline{H}_j \overline{L}_j) = \frac{(I S - Z)}{10} \text{ yrs} \]  
(2)

\[ \frac{1}{k} T_i : T_j = L_i \overline{H}_i : (L_j \overline{H}_j) = P_i : P_j \]  
(3)

\[ L_i \overline{H}_i : (L_j \overline{H}_j) = (\frac{FS}{Z}) : 10 \text{ yrs} \]  
(4)

In (1)-(4), the ratio \( L_i \overline{H}_i \overline{L}_i : (L_j \overline{H}_j \overline{L}_j) \) and \( L_i \overline{H}_i : (L_j \overline{H}_j) \) - respectively for volume and plane problems - is equal to the ratio of natural press (P) of the compared slopes, i.e. to the ratio of the main forces which determine the press-deformed state.

\( T \) is the period between two landslide process activities, rhythm volume with stages of activity damping, the transition to the new balance system, relative stability (G.I. Shahumyan's terminology).

\( k \) is the ratio of the speed of change of energy state of elements (i and j) of the bank slopes' geosystem in the period of \( T_i \) and \( T_j \). The values of \( k \) is a series of whole numbers in the intervals of 1 - 15, to 20.

\( H \) is the height of the water parting above the erosion basis;
\( L \) is the width of the slope between two erosion forms; \( l \) is the linear parameter of the intertravine block, normal \( L \).

In particular case \( l = B \) - the slope \( \tan B = H/B \ldots \) (5).

It can be showed that correlation (1) - (4) is common for a wide circle of geomechanic models including the models of the class considered above. E.g. referring to one littoral lot for \( t_1 \) and \( t_2 = t_1 + \Delta t \) intervals of stationary (quasi-stationary) stability regime we have (if according to (2), \( \Delta H = 0.11H_1 \); \( k = 1 \); \( L_1 : L_1 = 1 \);
\( \tan B_1 : \tan B_2 = 1 \). Hence (1), taking into account (5), changes into

(6): \( t_1 : t_2 = (H_1 : H_2)^2 \)

where \( H_2 = H_1 - \Delta H \), which is basic, as we know, in the theory of linearly-deformed systems' consolidation (G.I. Pokrovsky, N.V. Fyodorov, 1939. D. Tailor, 1960).

The expressions for limited (according to properties indices) states of ground systems are analogous in the works of N.N. Maslov (1957) and A.I. Ksenofontov (1964).

According to (1), the regularity is real in nature, that is the same periods \( T \) (but not the time of the stages) may have slopes with different \( H, L \) and \( B \) but in combinations \( (H \cdot L \cdot B) = \text{Const} \).
corresponding to approximate equality of littoral blocks mass and its integral gravitational influence upon the rocks or the level of erosion base. And since the abrasive banks of large basins have approximately identical plan curvature of interravine blocks [5], each theoretical T (with mass equality and in definite borderline conditions) has a series of maximum stable slopes with the single, i.e. hyperbolic, dependence between the height and slope:

\[(H \times B) = \text{Const.} \quad (7)\]

The expression (7) is close in meaning to the empirical dependence between H and B which was revealed by E.P. Emelyanova for landslide areas of the Russian plain and which is "the basis of the comparative method while evaluating the general stability of the slope" [5]. The geographical position is not included into the borderline conditions of dependences (7), (1)-(4). The sphere of their application is determined by the conditions of general geologic similarity such as plain territories, the development of sedimentary, mainly "loose" rocks, characterised by the commensurable statistics of properties; the intervals of recent historical time; geological bodies on large basins banks. Let us show the method of interregional geodynamic analogues can be used not only within the Russian plain limits but elsewhere, e.g. in the littoral zone of the English Channel near Folkestone-Warren. The parameters of the geomodel are:

- \( H_{FW} = 122 \text{ m}, \ L_{FW} = 3 \text{ kil}; \)
- according to observations since 1765
- \( T_{FW} \approx 18-20 \text{ years (K. Terstages, 1950)} \).

For the prognosis of \( T_{FW} \) let us use the parameters of the four analogues studied before, mainly interravine blocks of the Odessa coast - I, II, III, IV [5]. Respectively \( H(m) \) is 53; 41; 46; 44; \( L(\text{kil}) \) is 7.4; 1.75; 2.75; 6.45 and the average of many years since 1820 \( T_{od}(\text{yrs}) \) is 18; 60; 35; 28. Taking into account these data, according to (4), we have a series of \( k \), namely 1, 16, 5, 2, for \( T_{FW} \) calculation. According to (3), a series of theoretical values \( T_{FW} \) is 17, 19, 20 and 18 (on the average of 18.5 yrs) which coincides with the actual rhythm intervals. Moreover, the calculation of \( T_{FW} \) according to the derivatives (from the Odessa one) analogue, gives the similar result. E.g. for the Volga littoral block between the Polivnyanskaya and Strizhevskaya ravines near Ulyanovsk (\( L = 7.4 \text{ kil}, H = 120 \text{ m} \)) \( T \) is 12.2 yrs. Hence, according to (3)-(4), \( T_{FW} \) is 20 yrs. Let us note that the actual data (observations since 1865) \( T_{UL} \) is 12-14 yrs. The results need no comments.

Borderline conditions and assumptions of the method:
1. Dependencies (1)-(4) are only a link in one of the laws of correspondence in nature and the properties of the geological bodies which are described in them are the sub-
system in the hierarchic structure of geospace. E.g. in the morphometric series of V.V. Piotrovsky [11] uniting the geofoms of 18 orders from sandy ripples with LH in 1-3 cm² to the biggest tectonic structure with LH in (1-3) × 10¹⁷ cm² – the above mentioned mesofoms almost entirely belong only to the X order. The similar series of geofoms is elaborated by A. Kaye and Zh. Trikar (1959). Quantitatively expressed geometrical similarity of the forms of different order is determined by the effect of natural pressure and by the tendency to isostatic equilibrium which is partially "maintained by common properties of the slope elements and some constant rock properties such as the angles of natural slope, shearing, etc." [11]. The regularity is explained by the summary (according to resonance type) influence of the lunar-solar attraction upon the lithosphere of the Earth, alterations of the rotational regime of the Earth under the influence of endo-and-exogenous processes, climate indices' variations. Let us note that only rotation-pulsation phenomena connected with the solar activity change the potential of latitudinal deforming forces on 30-50 kg/cm² (M.V. Stobac, 1959). Hence, the quasi-stability of the fields of slopes stresses – the geodynamic system elements – is determined chiefly by the dependence of the field of local gravitational forces upon the general gravitational force of the Earth interacting with the fields of the cosmogeophysical background.

II. The calculation operations, according to (1)-(4), are real: 1) for slopes-analogues which differ in mass or volume forces of the geosystem not more than in an order; 2) for littoral slopes to a considerable extent consisting of the rocks which are capable of a viscous plastic flow of one or another type in the forced conditions of the oscillating contour type. The significance of this factor in deep landslides' formation, e.g. in that of the Odessa region, is well known. Let us show its significance for landslide flows in loess mass. E.g. there is the loess mass of about 10-12 m on the Dnieper banks near Kiev between the Syrets and Lybed rivers (H = 102 m and above the maximum level of the river H = 13.7 kil) which lies on marl and sandy-clay rocks base. The landslide flows were active there in 1971, 1877-1879, 1888, 1895-1897, 1905-1907, 1915-1916, 1924, 1933-1934, 1944, 1953, which determine the average of many years T_K in 9.1 yrs with variational coefficient of 0.1. Using the analogues "Odessa I and IV", "Folkestone-Warren" and "Ulyanovsk" we get, according to (3) and (4), a series of theoretical values for T_K. They are 9.2, 9.7, 9.1 and 9.5 (on the average 9.4 yrs) i.e. a result similar to the actual one. Below there is an example of the detailed analogues' des-
cription on the basis of complex modelling.

"The Sokolovaya Hill" is the type analogue of the Volga banks' landslides. Let us consider the landslide slope of the right bank of the Volga near Saratov which has a width (L) of 2.1 kil between the Makhannaya and Glebulcheva ravines and which was 133 m in height before the reservoir was filled. The Hill and its foot consist of clays with interlayers of aquiferous sandstones and sands (the Jura, Chalk, Cainozoic sedimentary rocks). The bedding is nearly horizontal, it is of the block type with rock deformations beneath the erosion base. Judging by H, the analogu belongs to the X order of the morphometric series [8]. According to the similarity conditions, analogues "Odessa I,III,IV", "Kiev", "Ulyanovsk", "Folkestone-Warren" belong to the same group. Using them we can calculate the most probable rhythm T_{a,of m.yrs}. At first, according to (4), we receive a series of "k" which is 2.9;1.14;6;2. Then, according to (3), we get a series of T_{a,of m.yrs} it is 24.4;26.3;26.4;26.2;26.1; 25.5 (on the average 26 yrs). According to actual data, the ground relaxation of the Sokolovaya Hill stresses (landslides with the volume of dozens of millions m\(^3\)) took place in 1783, 1818, 1846-1849, 1869, 1884, 1913-1915, 1927, 1968. Hence, the average of many years is 26 years with the variation coefficient of 0.34. The coincidence of the actual T_{a,of m.yrs} with the theoretical one is strikingly exact as we can see.

But the most important for understanding the essence of the landslide rhythm information revealing the post stages of the stressed state of the Hill can be obtained only with the help of laboratory models. In this respect such methods were used as the photoelasticity methods, electrogeodynamical analogues (EGDA) and the method of equivalent materials; The latter was used in the variant giving the opportunity to take into account the lowering of ground strength indices - model equivalents [7] - in time.

The most characteristic combinations and stress concentration zones were revealed by means of physical and analogue modelling methods. The stress values and their dynamics in the process of development of the landslide dislocation were defined more precisely with the help of the model using the ohmic resistance transducers. The modelling was carried out on the scale of 1:250. The height was 54 cm and the slope was 240 cm which determined the choice of the friction angle as the properties index, guaranteeing the observance of the mechanic similarity. Rock analogues of the foot of the slope were phlogopite-flake, Aptian-Barremian clays, and the dislocated blocks of Aptian sandstones were modelled by fine quartz sand with addition of
mineral oil. The modelling of the press equilibrium of the head water-bearing horizon was carried out with the help of "pneumocushions" connected with piezometers. Mineral oil was added.

The following results were obtained. 1. The fractures were formed in the upper part of the slope, in the zone of tension stress with horizontal stress (\( \sigma_x \)) to 0.5 MPa.

2. In the middle part of the slope, in Aptian sandstones and the underlying clays there was a zone of concentration of all kinds of stresses. The values of vertical stresses (\( \sigma_z \)) 1-2 MPa; the values of tangent stresses to 0.7 and \( \sigma_x - \sigma_z + 0.5 \) MPa.

3. In the foot of the slope \( \sigma_z \) reached 3-3.4 MPa; before the formation of the pressed out bank \( \sigma_x \) reached 3 MPa and lowered after the dislocation to 0.8 MPa.

4. The estimation of the slope's stability of the Hill before the landslide in 1968 showed that in a condition close to the breaking point (\( C_{st} \sim 1.1 \)).

5. The breach of the slope's stability took place due to the lowering of the lowering of the rocks' firmness in the lower part of the section (Barremian clays), in particular during the decrease of cohesion (10^3 Pa), from -40 to 5. The data obtained show the special character of the Sokolovaya Hill geocenose and give the opportunity to refer it to type natural analogues of landslide process regional prognozing. The analogues complex research was also carried out for the Odessa coast. To sum up the results of the research let us pick out the main information obtained by laboratory modelling for type analogues' characteristics.

The method allows to estimate the general state of the slopes' stability, to take into account the values of \( C_{st} \) along the known surface of the dislocation and potential unstability zones; to estimate the quantitative role of separate landslide-forming factors and the effectiveness of constructions preventing landslides. In particular, besides the influence of the rock weight it is possible to take into account the hydrostatic, hydrodynamic and equilibrium stress of underground water, jointings, the initial tectonic stress, etc. Special methods give the opportunity of modelling landslide-forming deformations of plastic flow. Nevertheless, the method does not provide solutions of real time volume of the development of the slope's deformation in definite conditions and the T rhythm volume. Moreover, the independent application of the model for geological-engineering zoning is very labour consuming on the banks of large basins with diverse natural conditions, e.g. on the banks of the Siberian basins. Preliminary prognostic recommendations elaborated on the cybernetic basis are necessary. That is why complex application

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of the model method and of the method and of the method of inter-regional geodynamic analysis is the most perspective. Taking into account the value of T and the natural conditions we can approximately but with sufficient precision for the practical aims differentiate the territories according to slopes' stability. So interravine littoral blocks with T of 100 years should be considered stable; with T of 50-100, 25-50 and 10-25 years respectively stable, stable enough and relatively stable.

The period of 10 years characterizes unstable geodynamic situation. In order to distribute the predominant height H and L we investigated, defined and united into the subsystems the cases of both theoretically stable slopes and the spectrum of the morphometrical conditions, which determine the display of T in a real situation in the intervals from 5 to 100 years. So the period of quasi-stationary regime in 100 years theoretically takes place for the series of littoral blocks with combinations: L(kil) - H(m): 1-30, 2-15, 3-10. For T 50 years the series is L - H : 1.5 - 62, 2-17, 3-31,4-23, 5-19; slopes with T 23 years have combinations L-H : 2-140, 3-93, 4-70, 5-56, 7.5-37, 10-28; the T rhythm of 10 years theoretically corresponds to values L - H : 5-270, 6 - 220, 8 - 166,10 - 133, 12-110, 15 - 90. As we see, the product of LH of each series is constant (of hyperbolic type). The branches of hyperbolas with T of 50 years practically merge for analogues when L = 0.5 kil - H = 200 m and H = 10m - L = 0.5 kil.

The littoral blocks either with steep surfaces of the erosion cuttings (more than 20°-30°) the stability of which is estimated with the help of the additional factors or very gentle slopes correspond to them. The significance of the combinations can be checked up by the method of reverse calculation. So the prognostic calculation of T_a. of m. yrs according to block series with T = 50 yrs determines, according to (4), k = 6 and according to (3), - T_a. of m. yrs = 26 years. The same result was obtained using the analogue series with T=10yrs.

The probable change of T and therefore the slopes' stability caused by the filling of the reservoir can also be quickly estimated (according to the ratio T_1 : T_1 + ∆T_1). The method can be used, of course, only after some years of the reservoir's exploitation when the process of reworking begins to damp and the successful processes of the geodynamic activity of the regions acquire more and more significance.

The complex application of the methods considered above is a simple and not labour-consuming way of receiving quick and practically important estimation of the geodynamics of big territories on various stages of their exploration both from the time and stability estimation positions.
REFERENCES


VII. 82
SOME NEW POSITIONS OF THE NATURE AND MECHANISM OF THE LANDSLIDE PROCESSES AND THEIR APPLICATIONS TO THE PROBLEMS OF WATER STORAGE SHORES STABILITY

QUELQUES POSITIONS NOUVELLES DE LA NATURE ET MECANISME DES PROCESSUS D'EOULEMENT DE TERRE ET LEURS APPLICATIONS AUX PROBLEMES D'ACUMULATION D'EAU DE LA STABILITE DE RIVAGE

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ABSTRACT

Intensive loss of rock strength on the water storage shores to the great degree is connected with reiterative changes of their stress conditions by the drawdown of water storages. With multi-layer stratum, deformation process in higher layers is accompanied by display of their dragging-along action on sublayers, because of structural bonds. This defines the mechanism of multi-layer landslides. The following schemes of mechanism are presented here: a) of multi-layer landslides, including cases on the water storage shores; b) resumption of old landslides activity on the water storage shores with deep reach or old masses in accordance with location of weakened zone, formed as a result of water storage work. Unknown and not taken into account previously the dragging-along action of rock was revealed basing on the results of the investigations as a new and one of the most important factors of landslides formation, which should be taken into consideration for predicting stability of multi-layer landslides and water storage shores and for appointing of counter-landslides measures. The value of dragging-along action of overlying strata of landslides mass to the underlaying rocks or landslides is a part of external shear force mobilized at the beginning of overlying sliding and limited by the value of rock shearing strength on the contact surface of carrying- and carried-along layers or landslides. Decisive role of force affect of underground water is shown as a result of investigations in manifestation of both surface and deep landslides in clay rocks. In some cases even with water encroachment only within a part of their strata. The ways of protection measures against landslides are shown here. Special importance has been estab-
lished of protection from water encroachment and thus stabilization of surface landslides is often sufficient for total slope stability.

La déconsolidation intense des roches sur les rives des retenues est liée dans une grande mesure avec les changements se répétants à maintes fois de leur état de contrainte pendant le marage des retenues. Lors d'assises multicouches des roches sur les pentes litorales, la déformation des couches susétendues est accompagnée par la manifestation de leur influence entraînante sur les couches sous-jacentes qui détermine le mécanisme des éboulements multi-couches. Sur la base des recherches on a reçu le schéma des mécanismes : 1) des éboulements multicouches; y compris sur les rives des retenues; 2) de recoinmenzement de l'activité ébouleuse sur les rives des retenues avec enveloppement des anciennes masses ébouleuses en conformité de la situation de la zone modérante, formée finalement de l'activité de la retenue. L'influence entraînante, des roches, inconnue et non-comptée auparavant, est révélée finalement de recherches en qualité d'un facteur nouveau, un des facteurs importants formant l'éboulement qui doit être compté pendant la prévision de la stabilité des éboulements multicouches et les rives des retenues et lors de la fixation des mesures antiébouleuses. La grandeur de la force entraînante des masses ébouleuses susétendues, transmettant sur les roches sous-jacentes ou sur les éboulements, se représente une partie de l'effort extérieur de glissement, qui se mobilise au début de la poussée de l'éboulement supérieur, se borne par la valeur de la résistance des roches à la dislocation sur la surface du contact des couches entraînantes et entraînées ou des éboulements. Au total de toutes les recherches on a montré le rôle décisif de l'influence de force des eaux souterraines à la manifestation des éboulements superficiels et profonds dans les roches argileuses et dans quelques cas pendant l'inondation les dernières dans les limites seulement d'une partie de leur puissance. On a indiqué les méthodes de la protection antiébouleuse. On a determine l'importance particulière de la protection contre l'inondation. C'est ainsi, la stabilité des éboulements superficiels est suffisante assez souvent pour la stabilité totale de la pente.

Presented in this paper propositions were determined basing on the achievements of modern Engineering Geology, on the author's theoretical research work and detailed analysis of the results of
many years of field observations and laboratory tests, which were carried out on a number of landslides in the USSR under the author's leadership.

Investigations of the sliding slopes showed that on most of them there are multi-layer slides of 2–3 and sometimes of 4–6 and more sliding bodies, separated by layers of weakened clay rock. Landslides of all layers can be divided into three groups: surface landslides — affecting the most weathered overburden deposits; middle slides — in subjacent more dense and overcrumpled clays from plastic to semi-firm consistence and deep landslides — affecting the bedrock or displaced blocks of solid rocks.

Long preparation of landslides in the process of weathering, accompanied by changes of stress condition and rock strength and also by deformations in creep phases precedes sliding actions.

First of all investigations revealed the processes of rock weakness and formation of weakened zones in them as a result of water pressure action, both on relatively considerable depth in clay-rocks contact zones with water-bearing horizons by multiple actions of hydrostatic heads and on the water-storage banks with multiple drawdown and in the surface deposits on the slopes with their multiple water encroachments and drying up, accompanied by multiple changes of rock shrinkage and swelling cycles and display of sign changing displacements.

On the slopes with the multilayer rock stratum (and also with stratification, depending on location of weakened zones) the layers deformation process along the slopes in the direction of fall is analogous to the development of soil deformations in deep footings of structures under the increasing vertical loading.

External forces change the initial (defined by the rock dead load) stress condition, causing the increase and redistribution of stress and displacements in rock. As the compressive stresses along the slope length increase, all three phases of deformations (compressive (elastic), plastic and progressive failure) can form independently and sequentially in every layer. Load increase leads to the increase of the second and third phases and reduction of the first one.

Investigations show special unfavourability of multiple force action of underground water, leading to the following reduction of rock strength on layers boundary in the process of sign changing displacement and accumulation of residual deformations as a result of multiple displacement and rock compaction. The clay rock structural bonds, distributing and equalizing the stresses in the rocks, carry out the dragging—along action of more stressed rock elements on less stressed by their relative displacement.

Sliding of slopes in most
cases begins from the upper more active landslides. After surface slidings come displacements of middle and deep slides with reduction of velocity with the depth, often with time lag.

The character and values of relative displacements of different slide layers and also pressure diagrams on the retaining walls, based on the results of investigations, indicate the mutual braking dragging-along actions of the adjacent layers and likewise in the creeping phases.

The nature of this influence consists in the work of the structural rock bonds of stresses distribution between differently loaded rock layers. With relative displacements of rock layers still in the creeping phases, the structural bonds transmit stresses from the most unstable, beginning to slide, layers to more stable underlying layers, creating in the latter stressed zones.

When acting stresses exceed strength of structural bonds, the latter break. This moment practically corresponds to the action of maximum dragging-along forces from rock layers beginning to shift under the influence of its own dead-weight and external shear forces.

The break of bonds on the layers boundaries as a rule occurs after long reiterative sign changing displacements, which lead to gradual lowering or rock strength. Sign change and multiple displacement are of practical importance and are connected with multiple changes of rock stressed conditions, first of all with the change of their water encroachment.

Thus the dragging-along actions and dragging-along forces are made by dragging-along layers, which can slide under the dead weight and external shear forces influence in the rock strata on the slopes relative to underlying and overlying layers. Their values are limited by rock strength on the contact of dragging- and dragged-along masses.

The value of the dragging-along force of the overlying landslides ground masses on the underlying rocks or slides is a part of external shear force, mobilized at the beginning of sliding and limited by the value of rock shearing strength on the contact surfaces of dragging- and dragged-along layers or slides:

$$ F_{dr} = \tan \gamma \cdot \text{cont.} N_1 + \text{cont.} L - T_1 $$

or (with $c = 0$)

$$ F_{dr} = \tan \beta \cdot \text{cont.} N_1 - T_1 $$

(See figure 1)

Promoting the rocks breaking off from the solid slope and their compression in the direction of sliding, the dragging-along action directly leads to the formation of deep cracks in the head of lower layer slides (beginning even in the creeping phase) and side hydrostatical pressure, when cracks are filled with water, and possible hydrostatic heads in the sliding zone.
Fig. I: Scheme of double-layer slide mechanism with the upper slide dragging-along action.
I - upper slide body; II - lower slide body; 3 - seepage flow (with hydrodynamic pressure $P_d$); 4 - sliding surface of the upper landslide; 5 - dragging-along force; 6 - the first breaking off crack; 7 - separated block of solid slope; 8 - the second breaking off crack; 9 - side hydrostatic pressure on the lower slide masses; 10 - sliding surface of the lower landslide.

It infers from the above said that the dragging-along action is a new slide forming factor, although the existing conception of multi-layer landslides mechanism and generally accepted methods of stability evaluation take no account of it, therefore this factor should be considered first of all in the analyses of multi-layer landslides mechanism and also similar disturbances of stability of water storage shores with participation of dragging-along forces.

Scheme of Multi-layer Landslides Mechanism
(Example of double-layer landslides with beginning of sliding in the upper layer. See fig.I)

Development of sliding process is going in the following order:
- development of upper slide masses (for example under hydrodynamic pressure action);
- compaction of sliding mas-
ses in the zone of compressive phase, in the head of upper and (under the influence of dragging-along action) lower slides, formation of the first fracture crack;
- breaking of seepage flow along the crack length, stopping of hydrodynamic pressure action;
- breaking off of the rock block from the solid slope at the head of slides, which lost the stop at the bottom. It is relieved by the sharp local increase of hydrodynamic pressure. Then follows closing of the first and formation of the second fracture crack;
- recovery of seepage flow in the upper slide masses after filling of the 2-nd crack with water; formation of the side hydrostatic pressure on the lower sliding masses;
- manifestation of hydrostatic heads in the zone of sliding surface of lower landslide with hydraulic connection of this zone with the second crack;
- repeated sliding of upper landslide;
- total displacement of the whole sliding mass along the lower sliding surface under the joint action of all forces.

The mechanism of sliding revolved activity on the water storage shores is analogous to the considered above (see fig.2).

Taking into account the dragging-along action considerably changes methodology of multi-layer landslides stability assessment.

General positions of Design Method

In contrast to the generally accepted statical scheme (which does not consider displacements of upper layers landslides and takes into account the total weight of the whole slide thickness up to the failure surface of the lower slide layer) for quantitative assessment of the slope stability, we consider the design scheme, according to the beginning of the upper landslide displacement, conditionally called as a kinematic scheme.

According to this scheme the effect of the upper landslide on the lower one is replaced by application to the lower slide, as external forces, of dragging-along force and weight of the upper slide.

The analysis shows that such a scheme enables to take account of the transmission (by the structural bonds) on the lower layer slidings of force actions manifested in the upper landslide layers.

These force actions are as a rule not taken into consideration in calculation for they are treated as internal forces in relation to the lower sliding surfaces.

According to the kinematic design scheme the total safety factor of the slope with multi-layer slides is as follows:

\[
K_{\text{kin}} = \frac{tg\beta G \cos \alpha}{G \sin \alpha + F_{\text{drg}} \cos (\alpha - \alpha)}
\]

(see fig.1)

\[
\max F_{\text{drg}} = \text{mob.} \cdot \text{He} \cos (\alpha_{y} - \alpha)
\]

\[
G = G_{1} + G_{2}
\]

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where
F-drg. – dragging-along force;
mob. He – a part of the external force, mobilized by breaking down of the upper landslide stability.

From the equality : \( K = 1 \)
mob. He \( \cos(\alpha_1 - \alpha_1) = \tan \gamma_1 G_I \cos \alpha_1 + c_{1l} = G_I \sin \alpha_1 \); and so
\( \max F_{drg} = \tan \gamma_1 G_I \cos \alpha_1 + c_{1l} - G_I \sin \alpha_1 \).

For the slopes with sliding beginning in the lower layers, when there are no displacements of upper layers in relation to lower layers, calculations should be carried out according to usual statical scheme.

Example of Calculation for the Slope with Double-layer Landslides (see fig.1)
The initial data:
- the upper layer:
  \( G_1 = 15000 \text{ kN}; \alpha_1 = 6^\circ; \gamma_1 = 7^\circ; \)
  \( c_1 = 5 \text{ kN/m}^2; \lambda_1 = 120 \text{ m}; \)
  \( D_1 = 1100 \text{ kN}; \alpha_2 = 8^\circ; \lambda_1 \cos(\alpha_1 - \alpha_2) = 11000 \text{ kN}; N_1 = G_1 \cos \alpha_1 = I4900 \text{ kN}; T = G_1 \sin \alpha_1 = 1550 \text{ kN}. \)
- the whole (upper and lower) landslide:
  \( G = 20000 \text{ kN}; \alpha_2 = 4.5^\circ; \gamma_2 = 8^\circ; \)
  \( c_2 = 5 \text{ kN/m}^2; \lambda_2 = 110 \text{ m}; \)
  \( N = G \cos \alpha_2 = 19940 \text{ kN}; T = G \sin \alpha_2 = 1570 \text{ kN}. \)

![Fig.2: Scheme of landslide resumption on the water storage shore.](image)

1 and 2 – breaking off cracks; 3 – separated block; 4 – the upper seepage flow (with \( D_1 \)); 5 – weakened zones of old landslide deposits in bouaries of the draw off; 6 – side hydrostatic pressure on the old slide masses; 7 – sliding surface of upper landslide; 8 – dragging-along force; 9 – the second seepage flow; 10 – sliding surface of the lower landslide and of the whole sliding strata; II – water storage draw off.

Side pressure of separated block \( E_a \cdot B \cdot \cos(\alpha_1 - \alpha_2) = 800 \text{ kN} \)
Side hydrostatic pressure \( H \cdot \cos(\alpha_2 - \alpha_2) = 1000 \text{ kN} \)

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Determination of safety factor for the upper landslide:

$$K_I = \frac{\tan^{2} N + c l}{T + D \cos (d_i - d_i)} = \frac{\tan^{2} 14900 + 5 \times 120}{1550 + 1100} \approx 0.92$$

Calculation of the value of mobilized part of external forces (in this case of hydrodynamic pressure) by stability limit (K=1) of the upper slide (and thus the maximum of dragging-along force).

$$\text{mob.} D = \tan^{2} (d_i - d_i) = \frac{\tan^{2} N + c l}{T} \approx 880 \text{ kN};$$
$$\max F \text{ drg.} = 880 \text{ kN};$$
$$\max F \text{ drg.} x \cos (d_i - d_i) \approx 880 \text{ kN}.$$  \text{Total}\ K:\n
a) calculation according to usual (statical) scheme:

$$K = \frac{\tan^{2} N + c l}{T} = \frac{\tan^{2} 14940 + 1200}{1570}$$
$$5 \times 110 \approx 2.1$$

b) calculation according to kinematic scheme (with F drg.):

$$K = \frac{\tan^{2} N + c l}{T + F \text{ drg.}} = \frac{\tan^{2} 14940 + 1200}{1570 + 880} \approx 1.37$$
$$+ 880 \approx 1.37$$

Thus the investigations showed earlier unknown and not taken into account rock dragging-along action x) as a new and one of the most important slide-forming factors, which should be taken into account for prediction of multi-layer landslides and water storage shores stability and also appointing of protection measures against landslides.

x) This has full analogy with a railway displacement under the traction force.

The absence of such accounting in a number of cases leads to un-
stability of slopes or to the increase of excessive expense for the counter-sliding protection.

Taking into consideration the given above explanation of the deep multi-layer landslides, the results of the carried out investigations made it possible to solve many years problem for many large slides in clay rocks, concerning the role of the underground water in the sliding manifestation including slidings on the well known Ulianovsk and Batraki hill-slides.

In this aspect special significance is acquired by our investigations on a number of slides in clays and clay rocks carried out in the various regions of the USSR of quantative estimation of the hydrogeological factor in preparation, formation and development of landslide processes.

In this view investigated were the landslide slopes with water supply practically exclusively by precipitation (in Suchumy, New Aphony and other landslides on the Black Sea coast of Caucasus) and otherwise the landslides with water supply by precipitation and cropping out or embedded in landslide masses of water-bearing layers (in the Volga region).

On the slopes with acting or acted landslides the investigations of the role of the hydrogeological factor were fulfilled in two variants: on the unstable landslide slopes and on the previously sliding sections fixed with the application of retaining walls.

The analysis was carried out on the basis of many years observations of slide displacements on different depth (from 0.5 - 1.0 m to 25-30 m) in the horizontal and vertical planes.

These investigations were combined in time with observations of water balance components of slide masses, such as precipitation, inflow of surface, underground and industrial water, surface and underground run-off and evaporation, and with calculation of water balance and with the analysis of the resulting water accumulation in the slide body and moisture content.

The main results are as follows:

- determination of the slidings cyclical character, corresponding usually to the cyclic changes of the amount of water contained in the sliding masses (first of all owing to the precipitation);
- corroborations of the hypothesis about existence for each studied landslide type of its own critical amount of water, accumulated in the slide masses on the slope, exceeding of which directly leads to the sliding displacement. So, for a number of landslides on the Black Sea coast of the Caucasus in the so called homogeneous clays without defined horizons of underground water and with basic water supply by rain-fall water, the critical amount of accumulated water corresponds to 150-180 mm column of water, distributed along.
the whole area of the landslide;
- determination of nature of critical amount of water and its correspondence with the critical hydrodynamic pressure (of water filling the cracks and other voids in slide masses), which determined breaking of slide masses stability.

**Investigations on the Stabilized Slope with the Use of Retaining Walls (fig.3)**

These investigations include many years observations of the landslide masses pressure on the retaining walls (with the help of special sensors mounted on the back surface of the walls).

![Diagram of landslide investigations](image)

**Fig. 3:** Experimental investigations of the pressure to retaining walls on the landslides.

a) scheme of the retaining structures and of the sensors location; x - the place of the sensors installation; c) diagrams of the pressure, precipitation and ground moisture content (moisture content data is insufficient - only for two holes);
b) diagram of experimental values of stresses on the back side of walls.

At the same time synchronous observations of precipitation, water level and moisture content in the slide masses on the slopes and behind the walls were carried out.

In accordance with the problem of determination of the hydrogeological factor was ascertained.

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the following:

- cyclic character of changes with the time of both total and landside proper pressure on the walls with values from about 50 kN on one running metre of wall length in the dry summer seasons to 200 kN and more in periods of winter rains. (The depth of walls laying was over 8.0 m and of sensors - up to 6.0m);
- practical coincidence of cyclicity with the time of pressure on the walls and precipitation changes;
- considerably small values of the shearing strength parameters in the weakened zones (on the sliding surfaces) of all slides, especially of upper layer (approximately 3 to 10 kN/m²), approaching the residual strength of the clay rocks, forming sliding masses;
- small changes (till practical absence of changes) of shearing strength parameters in the zones of sliding surfaces under the influence of changes of water encroachment degree of sliding masses and consequently, small influence of these changes on the slope stability.

Besides the described above investigations on the stabilized and unstabilized slopes, in order to make comparative quantitative assessment of different slide-forming factors and protection measures against landsides, there was used specially worked out methodology, which included fulfillment of a number of inverse cal-

ulations of slopes stability with taking into account different factors.

The calculations showed the greater influence (over 50% on the slides under consideration) of hydrodynamic and weighting pressure and on multi-layer landsides - of combined manifestation of dragging-along action and side pressure (of hydrostatic or hydrodynamic and separated rock blocks) in the head of lower landslide.

Thus, the results of all investigations showed obvious decisive role for the most landsides of investigated types of free water force action, which is accumulated by sliding masses and moves in them along numerous cracks as a result in the first place of hydrodynamic and weighting pressure.

The results of investigations made it possible to suggest in the applied part a new method of assessment of multi-layer landsides stability and also working out of measures of protection against landsides. Herein has been ascertained the first-rate necessity of surface slides on the slopes stabilization with proper protection from water encroachment not only for stopping surface sliding, but in a number of cases for ensuring total slopes stability with deep landsides.

Special role acquires protection of sliding masses on the slopes surfaces from the shrinkage.

As one of the possible and basic decisions is recommended
a complex, which includes a wide-range of agrosilviculture measures, special anti-shrinkage coverings of sand, local and vegetation soil, light water-ways. Scheme of such decision is shown on figure 4; it is elaborated according to the suggestion of the author for one of the landslides in so-called homogeneous clay on the

![Diagram](image)

**Fig. 4**: Measure complex for the protection of slide masses from the encroachment of surface and rain water. 1 - anti-shrinkage coating from vegetable soil, soil and local ground with grass sowing; 2 - planting of trees; 3 - planting of shrubs; 4 - turf; 5 - the main reinforced concrete shoot (on the calculated cross-section); 6 - brench telescopic shoot of reinforced concrete or wood with distance at 10-15 m; 7 - sliding masses.

Black Sea coast of the Caucasus.

The received conclusions and recommendations are especially important for prediction and ensuring of long stability of water storage shores, as display of surface landslides on them can in a number of cases bring to deep breaking of rock stability, even if there are coast-protecting and retaining structures at slopes footings.

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PHOTO-INTERPRETATION STUDY FOR HILL SLOPE STABILITY AND THEIR EFFECTS ON NATURAL LAKE FORMATION—PARTS OF U.P. GARHWAL HIMALAYAS, INDIA

ÉTUDE DES PHOTO AÉRIENNES POUR LA STABILITÉ DE COLLINE ET LEURS EFFETS À LA FORMATION DU LAC NATUREL—PARTIE DU GARHWAL-HIMALAYAS DE L' U.P., INDE

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ABSTRACT
Photo-interpretation study was taken up for hill slope stability analysis in U.P. Garhwal Himalayas, India. Slope failure in this region had given rise to formation of few natural lakes in the past. Gohna Tal in district Chamoli, U.P. is one of the famous examples formed due to damming of Birahi Ganga river by a heavy landslide in 1893. The other natural lakes existing in the area were Gudiyar Tal, Tarak Tal and Cheena lake. Photo-interpretation study has helped in recognising the sites of existing old lakes and probable cause of slope failure which resulted in the formation of these lakes. A number of active, old and potential slide zones have been identified. An attempt has been made to demarcate the areas where future possibility of lake formation may occur due to damming of river course by landslide mass.

ABSTRACT
de la formation du lac en futur due an barrage creé par le glissement de terres an course de la rivière.

INTRODUCTION

In Himalayan region landslide phenomena are wide spread. These pose serious problems for engineering construction. The hazards caused by slope failure may be reduced to some extent if the prone-ness of a zone to sliding may be known in advance. Aerial photo study has proved to be most advantageous in identification of landslide over large areas in rugged Himalayan terrain.

The present study was carried out in connection with geoengineering survey for hydroelectric projects in district Chamoli, U.P., India covering an area of about 450 sq.km. between Chamoli township and Helang village. Large format size aerial photographs on 1:20,000 scale were used. In the past the region witnessed heavy landslides by which some lakes were formed. Presently the lake beds are dry. The present paper highlights the possible causes of slope failure, which resulted in the formation of these old lakes as well as probable sites of lake formation in future due to blocking of river course by landslid mass. Aerial photo study with selective field checks has helped in categorising different mass wasting processes into active, old and potential slide zones (Fig.1).

The rocks exposed in the area are slate, dolomitic limestone and quartzite of Garhwal Group of rocks. These are thrust upon by Central Crystalline rocks along Main Central Thrust. Structurally the area forms a doubly plunging anticline.

ON PAST LANDSLIDE OCCURRENCE AND RESULTING LAKES

The area of study presents interesting case histories of lake formation due to slope failure. Aerial photo study has helped in recognising these old lakes. The sites of existing old lakes were interpreted on the basis of unusual dimension of valley floor, light photo tone of accumulated silt and presence of old slide zones in the vicinity. Besides the famous Gohna Tal, mention may be made of Gudiyar Tal, Tarak Tal and Cheena lake, which are described below :-

Gohna Tal :-

Located near Durmi village in Birahi Ganga valley, the tal was formed by the famous rock slide of September, 1893. This slide brought down enormous quantities of material and blocked the river channel forming a vast natural dam. Accumulation of water behind the dam resulted in the formation of Gohna Tal. The naturally formed dam was finally breached in July, 1970, which caused extensive
flooding downstream. The lake does not exist at present.

Gudiyar Tal :-
It was known to exist along Gudiyar Gadhera at a distance of about 3km from its confluence with Birahi Ganga-river (Fig.2). The tal was originally 800 metres long, and was formed due to damming of Gudiyar Gadhera by a landslip. Subsequently, during a second slip in 1868, the originally formed natural dam was destroyed. This resulted in a serious flood in Alaknanda valley. As the available literature does not indicate slope failure direction, the author after studying aerial photographs with selective field checks, postulates the following views :-

In the past when the tal was not formed, Gudiyar Gadhera was not flowing through the present sinuous course, but it must have had a straight course as indicated by the presence of lineament adjacent to the course. This lineament also marks the limit of left bank of former tal and might have controlled the course of river in the past. The tributary stream on left bank, which was joining Gudiyar Gadhera at the dammed portion, brought enormous quantities of fan material (presently stabilised). As a result of progressive encroachment of the then fan, it pushed the original course of Gudiyar Gadhera towards its right bank. Later on, due to under cutting of right bank by toe-erosion where presently a steep scarp is seen, the landslide took place and dammed the earlier course of river. Presence of old landslide scars and debris on the right valley slope near the outlet of tal indicate that failure of right valley slope was the main cause for damming the river. The present sinuous course of Gudiyar Gadhera was a later development when the natural dam was finally breached in 1868.

Tarak Tal :-
Three lakelets like depressions are observed near Tarak Tal village. These lakelets are in close proximity and are formed at an exceptionally high altitude along tributaries of Begar Nala (Fig.3). Though no firm opinion regarding the formation of these lakelets has been given so far, these may partly be the result of solution collapse in limestone and partly due to slump action. The following points may be suggested in supporting this view:

i) The country rock is mainly made up of dolomitic limestone, which is amenable to Karstification.

ii) Sinkhole like depression of one of the lakelets suggesting solution activity.

iii) Presence of old slump scar around Tarak Tal village.

iv) Localisation of three small lakelets within a distance of about 250 metres on a sloping terrain.
It is thus felt that these lakelets were originally the result of solution collapse in limestone terrain modified as slump lakes with accompanying solution activity.

**Cheena Lake**

An old lake referred in literature as Cheena lake (Bhatt, 1981) along with two lakelets was known to exist along Patalganga river. The location of these lake sites are not clearly mentioned. The Cheena lake was about one km. long and half km. wide. Definite clues regarding the existence of Cheena lake are not very much convincing from aerial photographs. However, if the lake was at all existing, it must have been along Ganesh Ganga river. The probable site is a km. stretch extending upstream from the bridge on this river near Darmi1 village (Fig. 4). At this bridge site the river course suddenly becomes narrower suggesting probable natural dam site, while the upstream portion is comparatively wider reflecting light photo tone of accumulated silt material behind the dam and hence suggests a former lake bed. The other lakelet like feature is seen about one km. south east of Darmi2 village i.e. further upstream along Ganesh Ganga river. No signs of other smaller lakelet in the vicinity of Cheena lake, are seen. The Cheena lake and other lakelet were formed as a result of blocking of Ganesh Ganga river by landslide mass. This slide originating from higher region above Ganai village had moved both towards Semkora Nal and Ganesh Ganga river but major part of the slipped mass came down suddenly towards Ganesh Ganga river, thus blocking the river course and subsequently giving rise to formation of Cheena lake. Presence of old slide debris over which Ganai village is situated and narrow course of river near bridge also favour this view.

One more old lake has been observed along right bank of Semkora Nal. Failure of right valley slope probably resulted in the formation of this lake.

**POTENTIAL SLIDE ZONES**

A number of active and potential landslide zones confining mostly to limestone and slate units, have been mapped in Birahi Ganga and Alaknanda valleys.

In the area of study landslides have been caused due to various reasons but the action of seepage water aided by gravitational force are observed to be the dominant factors. The evidences of these are well pronounced in Birahi Ganga valley and adjoining areas between Birahi Ganga and Karmnasa Nal. Besides, the impact of lithological character and structural features like faults, fractures, joints, structurally weaker portion of anticlines, superimposed folds etc. have resulted in crushing and crumbling of lithounits making them amenable to mass movement. It is likely that accumulation of stress
along the "Main Central Thrust" might be taking place in the core of main Piplikoti anticline and the accumulated stress when released may be giving rise to slides (Srivastava, 1971).

One of the most common causes of landslides in the area lies in undercutting of the foot of slope either by natural or artificial interference. Good examples of bank erosion are seen along Birahi Ganga, while in Alaknanda valley human interference has disturbed the angle of repose by constructing road through old slide mass as seen between Pakhi and Belakuchi villages. In higher reaches, frost action and solifluxion have specifically intensified sliding.

Taking into consideration the causes responsible for activating the slides in the area, an attempt has been made to demarcate the slopes which may fall in future. Most of the potential slide zones are located on south facing slopes which are generally unvegetated and are undergoing severe erosion. Aerial photo study helps advantageously to identify the factors controlling potential landslide zones e.g. nature of slope, rate and intensity of erosion over a slope, concentration of lineaments, amount and orientation of structurally weaker planes in the direction of slope, presence of springs, water falls etc.

Some of the prominent potential slide zones as mapped along Birahi Ganga and Alaknanda valleys are discussed below:

Birahi Ganga Valley
The prominent zones where possibility of slope failure may mostly be as a result of bank erosion causing toe erosion of the slopes, are observed opposite to Garigaon, Sainj, Nijmulla and Pagna Malla villages on right valley slope. Further undercutting by river may lead to slope failure.

About 2 kms. upstream of Birahi Ganga river from its confluence with Alaknanda river, the slope exposed on right bank forms a steep scarp of about 600 metres height with slope angle ranging from 60° to 80°. Besides, Birahi fault also passes in the vicinity of the scarp on its south. The river Birahi around the site has a sinuous course of flow. During bankful stage, the river Birahi strikes the right bank with great erosive force, resulting in undercutting of the same. This will be further aggravated by lubricating action of water during rainy season. The gradual undercutting of the right bank may result in slope failure, specially where river makes a sharp loop on its convex part.

Opposite to Garigaon and Sainj villages, on right valley slope, besides bank erosion, the situation may further be aggravated due to the presence of high intensity folding, fractures controlling the course of tributary streams and orientation.
of bedding planes towards tributary stream. In addition to this, already existing active slides of small magnitude within the zone may grow in size. Opposite to Garigaon village, a fault running across Birahi Ganga river in NNE-SSW and dipping in east with 60° may be an additional factor. Since the country rock is mainly made up of limestone, percolating rainwater along the structurally weaker planes may give more impetus to solution action resulting in slowly deteriorating the strength of rocks.

The adjacent area of Gauna village is yet not fully stable and may fail in future. Alternate sequence of dolomitic limestone and graphitic slate, presence of a fault along Birahi river, concentration of fracture trends, highly folded rocks in the vicinity and moderate to steep dip towards tributary streams are apparently the reasons (aided by climatic and denudational factors) for slope failure around Gauna village.

The old slide mass on right bank of Birahi river in east of Gauna village is partly covered with vegetation but many rills and gullies are developed on its river facing steeper slope. Due to the presence of springs and seepage, creeping is a continuous phenomenon in the area, which further aggraviates during rainy season.

In the west of Gauna village, a notable old landslide zone is observed. Here, Dolomitic limestone with carbonaceous and graphitic slate bands dip in NE, i.e. almost towards tributary valley, running in north-south along a fracture. Besides, two small magnitude active slides have already come up within this zone, which may enlarge in size in future. These conditions are quite sufficient for activating the old slide zone.

At about 500 metres east of Pagna Malla village in upstream direction, dolomitic limestone forms a steep scarp of about 400 metres height with average slope of 70° and dips in SE i.e. almost towards the river with amount varying from 46° to 56°. Considering the angle of friction (ϕf), at which under gravity sliding would generally occur, as 45° and angle of bedding i.e. dip as 'ζ', if ζ < ϕf, the slope could stand any angle upto 90° but if ζ > ϕf, then gravity would induce movement along the bedding plane and the hillside would not exceed the critical angle (Cooke et.al, 1974). Since here dip amount is greater than 45°, there are every chances of failure along bedding plane. In addition to this, rocks at this site are highly folded and active zones of landslides have started growing up on this slope.

Near Irani village, Birahi river is controlled by a fault, which displaces Chamoli quartzite. These quartzites exposed on right valley scarp face, dip in NE i.e. upstream side with an amount of
22° to 30°. Considering the under-mentioned odd factors, the right valley slope around the scarp may fail in future.

i) Main Central Thrust crosses Birahi river near this zone.

ii) Birahi river is controlled by a fault running in ENE-NSW direction.

iii) The cracks are found to be developed in river bed indicating tectonic activity.

iv) One of the prominent joint sets in quartzite dips in SSW i.e. almost towards the river, with amount varying from 70° to 80°, joint plane failure and slippage of rock masses are possible.

v) Break in topography is observed along a linear feature on right valley slope in cultivated land on quartzites of Irani village. This also marks the upper limit of potential slide zone.

At this site Birahi river before meeting Gudiyar Gadhera passes through a narrow gorge for some distance. If the right valley slope is affected in future, there is every possibility that the slide mass may dam the course of Birahi river and may result in the formation of a natural lake.

At the confluence of Birahi river and Gudiyar Gadhera, the left valley slope of Birahi river is affected by recent landslide. This slide zone is seen to extend along right bank of Gudiyar Gadhera for some distance in upstream direction. Adjacent and above the slide-effected area, old landslide debris are seen on right bank of Gudiyar Gadhera in upstream side. Though the old slide debris is presently stabilised due to thick forest cover, yet it may be prone to sliding in future, in view of above mentioned active slide-effected slope just below it, and may block the course of Gudiyar Gadhera temporarily.

Alaknanda Valley

The prominent potential slide zones in this sector are observed between Pakhi and Belakuchi villages. Besides, a number of active slide zones of varying magnitude, mainly the Tangni landslide zone, are located in this sector. The presence of active slide zones is attributed to Pakhi-Tangni fault zone, which extends from near Pakhi to Sialgad in a NE-SW direction and shows steep north westerly dip (Krishnaswamy et al, 1975). The material within the fault zone is highly shattered and pulverised.

The presence of fault zone and existing active slides may further aggravate the sliding phenomena. Besides, the general dip direction and amount coincide with the slope direction and amount at places. Since the amount of dip in this region varies from 18° to 60°, and generally is moderately dipping, the possibility of rock failure along bedding plane may not be ruled out. One more aspect which has accelerated sliding in this sector is man-made cause of widening the road. This has disturbed the repose con-
dition of slid mass of active as well as old slide debris. The slid mass from the existing slides gets mobilised during rainy season and further acts as an erosive agent on slope. The calcareous unit present within the dolomitic limestone and slates are prone to chemical weathering and may further act as catalyst in accelerating the slides.

During heavy rain fall conditions, these slides originating from higher region with huge amount of debris may block the course of Alaknanda river near Pakhi village and may result in forming natural lake for a shorter period.

Patalganga Valley
A number of active landslide zones are observed in Patalganga valley. These slide zones are generally concentrated along right and left bank slopes of Ganesh Ganga and Semkora Nal respectively. This may be due to the presence of a fault running along Ganesh Ganga river in NW-SE on one side and 'Main Central Thrust', passing over villages like Molta and Mamolta on other side. Since the zone is in close proximity to fault and thrust, along which movement may readily take place and the rocks are crushed/weakened, the area is highly susceptible to sliding.

Considering the presence of fault running along Ganesh Ganga river, slate exposure on left bank around Darmi, village dipping towards river, presence of old slide debris around Darmi and Ganai villages, presence of springs above Ganai village and active zone of sliding between fault and Main Central Thrust, any of the valley slope of Ganesh Ganga river may be affected in future and may block its course near the bridge site, thus resulting in formation of a lake.

CONCLUSION
From the foregoing discussion, it is visualised that many parts of the study area are quite susceptible to sliding. Aerial Photo study has helped in recognising such potential slide zones. In some cases, slope failure may block the river course resulting in lake formation in future. Protective measures should be taken in such cases. A critical analysis indicates that structural and geomorphological set up of the region influenced by lithological characters are the prime factors for slope failure.

ACKNOWLEDGEMENT
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REFERENCES


Legend to Figures

D - Mylonitic schist and gneiss with amphibolite and calc-silicate rocks.
C - Quartzite
B - Dolomitic limestone and slate with subordinate quartzite
A - Central crystalline
1 - Active landslide
2 - Old landslide
3 - Potential landslide
4 - Fault
5 - Thrust
6 - Major joint/fracture
7 - Lithological boundary
8 - Old lake
9 - Alluvial fan
10 - Attitude of formations
11 - Spot height in metres
12 - Probable site of future lake formation.

VII.105
Fig. 2  Map showing Gudiyar Tal along Gudiyar Gadhera and probable site of lake formation in future.

Fig. 3  Map showing Tarak Tal along tributaries of Begar Nala.
Fig. 4  Map showing Cheena lake and other lakelet along Ganesh Ganga river.
ASSESSMENT OF THE CAUSES AND CONSEQUENCES OF SLIDES ON THE SLOPES ENCOMPASSING THE PARAIBUNA/PARAITINGA LAKE, BRAZIL

GLISSEMENTS DE TERRAIN SUR LES VERSANTS DU RÉSERVOIR DE PARAIBUNA/PARAITINGA, LE BRESIL

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ABSTRACT
Almost 3,000 slides of different magnitudes affect the slopes encompassing the Paraibuna/Paraitinga lake leading on problems which involve its siltation, erosion of private-owners lands and roads surrounding it.

The methodological approach to the assessment of the causes of the slides took into account that they are aspects of the natural geomorphological evolution of slopes; this evolution was modified when the lake was made.

Depending on the geological, geomorphological, pedological and land use characteristics the slopes show different behavior to the lake actions.

ABSTRAIT
Le réservoir de Paraibuna/Paraitinga a une surface de 206 km². Ses versants ont été affectés par à peu près 3 000 glissements de terrain, ce qui a posé des problèmes d’ensablement, aussi bien que de dégâts de sols à des propriétés privées.

La méthodologie d’études était axée sur la considération que ces glissements bien que liés au remplissage du lac des barrages, devraient être considérés dans leur cadre naturel d’évolution géomorphologique.

Les prévisions à propos de l’évolution des glissements ont donc été faites à partir des données géologiques, en ses rapports avec la géomorphologie et encore de la distribution des différents types de sols. Sur ce panorama du milieu physique naturel on a analysé les interférences dues à l’occupation et à l’aménagement du territoire.

1. Introduction
The man-made lake of Paraibuna/Paraitinga located in the state of São Paulo, Brazil, is under the management of the hydroelectric company of this state, the “Companhia Energética de São Paulo – CESP”.

The consequences of almost 3,000 slides affecting the slopes encompassing the lake are:

a) the degradation of the slopes involving the erosion of private-owners lands, roads and foundations of other civil engineering works as bridges;
b) the siltation of the lake due to the accumulation of sediments derived from the slides. This siltation interferes in the loss of usable lake storage and in the behaviour of the Paraibuna power-house equipments.

The study of slides carried out by IPT’s staff was aimed at
clarifying their causes and consequences, taking into account that the siltation material may be derived from slides and from rivers contributions.

2. The lake and its catchment area

The lake is formed by two dams and seven dykes located at the Paraibuna and Paraightingarivens, mainly to regulate their discharges and secondarily to produce hydroelectric energy in the Paraibuna power plant.

The intercommunication of the rivers to get the filling of the lake was made by a channel during a short period of 36 hours against 3 months as projected. This intercommunication led to a 3 m drawdown in the Paraibuna river and a 15 m raising of water level in the Paraibuna river, obviously accompanied by slopes-instabilities.

The lake has a total storage capacity of \(5.10^8\) m\(^3\) and a usable storage of \(3.10^7\) m\(^3\), occupying an area of 200 km\(^2\) and having 800 km of perimeter.

The catchment area of the lake is situated over migmatites, granites, schists and quartzites. The relief is diversified ranging from gentle hills to mountain ridges controlled by lithology and structural aspects. The drainage has mainly a trellis pattern developed along joints and faults systems.

The rainfall in the area ranges from 1 500 mm/year in the Paraightinga hydrographic basin to up 2 500 mm/year corresponding to the hydrographic basin of the Paraibuna river.

The more extensive land use of the area started in the XVIII century and it was related to coffee agriculture. Later the most significant degradation of the landscape occurs in the 50's carried out by the deforestation associated with the industrial use of the wood as energy source.

b) those events acquired the today appearance after the arising of the lake.

Before the arising of the lake, the geomorphological evolution of the area was acquired by natural processes, accompanied or not by the effects of human activities. This evolution resulted in different shapes of slopes formed by different rocks and differentially occupied by man’s activities.

Taking into account those differences it were defined the "Homogeneous Sectors of Analysis": areas of the slopes encompassing immediately the lake presenting the same behaviour to the lake action in terms of instability. This action is due to:

a) the daily and direct erosive impact of waves on the slopes;

b) the oscillation of the lake's water level, depending on the season of the year or expresses episodes like the intercommunication of the two rivers to get the filling of the lake.

Those "Homogeneous Sectors of Analysis" were taking as individual entities in the assessment of the causes of slides and in the choice of areas to further stabilization studies.

3.2. Study Techniques

Working upon aforementioned premises the following programme of study was developed.

The areas of the hydrographic basins were covered utilizing the existing road network in order to obtain systematic observation and data collection with respect to lithology, landforms, geomorphological processes, land use and pedology. Those studies were developed in a reconnaissance level based on existing maps and aerial photographs.

Later, an area of 2 000 km\(^2\) surrounding the lake were mapped in a 1:50 000 scale in terms of land use and geomorphology. The mapping was accompanied by lithological, rock structures and pedological observations.

The geomorphologic study was aimed at characterizing the origin and degree of activity of the landforms and related processes in order to compose a natural picture.
of the different factors of surface dynamics and their relative weights acting on the stability of slopes.

The present land use study together with the past history of such usage permitted the evaluation of the interference of human activities on the physical environment, establishing some conditions concerning the slopes stabilities.

Finally, based mainly on geomorphologic informations and secondarily on lithological and land use data were defined the "Homogeneous Sectors of Analysis" as referred. Each one of them were described in terms of landforms, land use, lithology, rock structures, geomorphologic processes, diagnoses of slopes instabilities, and prognoses about their development.

Concomitantly it was carried out the sampling of the sediments deposited at the bottom of the lake, specially in front of certain great slides.

The sampling consisted of the installation of topographic orientational marks and the collection of deposited material with "Van-Veen" and "Piston Core" samplers. The former was used for shallow sampling (lower than 20 cm of depth) while the "Piston Core" was used for deeper sampling, around 2.0 m of depth.

The samples were described and tested in all details such as colour, sedimentologic and stratigraphic aspects and granulometric analysis.

4. Results Obtained
4.1. Slopes Instabilities

The slopes instabilities are due to different causes, acting to gether or not, such as geologic, geomorphologic, pedologic, climatic and land usage as resumed in Table 1. Those causes may be divided into:

- predisposed: related to the physical environment where the slide occurs, without the interference of land use factors.
- effective: the assemblage of elements directly responsible for the unchained of the slide, including the land use and physical environment factors.

The effective causes may be subdivided into preparatories, those causes acting on the preparation of the slide and immediates, those causes acting on the slide occurrence.

The evolution of the slides may be divided into different phases:

- Phase 1: during this phase occurs the individualization of blocks of soils in the upper part of the slopes, corresponding to the colluvium or residual soils. This individualization is due to erosive impact of waves on some geomorphic features such as "pied de vaches", ravines, gullies or some pedologic features such as galleries and fracturing of the soils.
- Phase 2: the blocks individualized during phase 1 are released by the waves and gravity actions provoking the overhanging of the upper part of the slope.
- Phase 3: during this phase that overhanging process is increased by the erosive impact of the waves on residual soils having low cohesion characteristics.
- Phase 4: during the phase both residual soil and the rock are submitted to the lake action, acquiring the slides their greatest development.

In the Paraiba/Paraitinga lake it was identified 2780 slides. From this total, those corresponding to a volume of slidied material greater than 5 m³ were mapped in detail including their geometric aspects. According to the sliding plane and to the mechanism of their evolution, the slides were classified in four basic types:

- type a: sliding plane passing through the colluvium. Generally are shallow slides corresponding to the phases of evolution 1 and 2;
- type b: sliding plane developed along residual soils: related to the phases of evolution 2 and 3;
- type c: rocky sliding plane in well developed foliation rocks. This type of slide is related to the phase of evolution 4;
- type d: rocky sliding plane in granites and migmatites. They also are related to the phase of evolution 4.

The Figure 1 is a part of the final product of the studies showing different "Homogeneous Sec..."
tors of Analysis", as well as the location of distinct types of slides and geologic, geomorphologic and land use data.

In order to obtain a quantitative parameter of the development of slides at each "Homogeneous Sector of Analysis" it was defined the "Index of Tendency" (I). This index was employed in the choice of areas to further stabilization studies depending on the interference of slides on the environment.

The "Index of Tendency" was determined by the formula:

\[ I = \frac{N \cdot V}{P \cdot n} \]

Where,

- \( N \) = number of slides at each "Homogeneous Sector of Analysis"
- \( P \) = perimeter of each "Homogeneous Sector of Analysis"
- \( V \) = volume of the slid material at each "Homogeneous Sector of Analysis"
- \( n \) = number of slides at each "Homogeneous Sector of Analysis" whose volume of slid material was determined by geometric measures.

The "Homogeneous Sectors of Analysis" (HSA) were classified in three categories according to the "Index of Teadency":

- \(< 600 \) low I, II, III, VII, XIV
- \( 600-3000 \) medium I, IV, VIII, IX, X, XII
- \( > 3000 \) high V, VI, XI, XIII

4.2. Silting of the lake

The results of the sampling of sediments at the bottom of the lake in front of slides showed that the material is deposited according to its granulometric characteristics: the coarser sediments are deposited nearest the slope while the finer ones are deposited farther, in a distance lower than 500 m.

The total volume of sediments carried by the slides to the bottom of the lake is \( 1.6 \times 10^6 \) m³.

The most important carriage of sediments to the lake is related to the whole drainage of the hydrographic basins, mainly the headwaters, with the deposition of coarser sediments at the mouth of principal tributaries and the finer ones being carried, in suspension, at greater distances.

The total volume of sediments carried to the lake by the headwaters is \( 47.10^6 \) m³.

The total volume of silting of the lake is \( 48.6 \times 10^7 \) m³ deposited in areas corresponding to the not usable storage capacity of the reservoir.

Taking into account the total storage capacity of the lake (5.10⁹ m³) just only 1% of that value is exposed to the silting.

5. Conclusions

a) The methodology employed in the study led to the individualization of the "Homogeneous Sectors of Analysis" as areas of the slopes encompassing the lake of Paraíbuna/Paraitinga having different behaviour to the lake action. Those "Homogeneous Sectors of Analysis" were used in the assessment of the causes and consequences of the slides. They were used too in the choice of areas of slides to further stabilization studies.

b) The lake action on the slopes are differentiated, depending on their geomorphologic evolution, land use, lithology and rock structures.

c) The silting of the lake due to sediments derived from slides or from the hydrographic basins is not significant, taking into account the total storage capacity of the reservoir.

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<table>
<thead>
<tr>
<th>CAUSES</th>
<th>GEOLOGIC</th>
<th>GEOMORPHOLOGIC</th>
<th>PEDOLOGIC</th>
<th>CLIMATIC</th>
<th>LAND USE</th>
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<tbody>
<tr>
<td>PREDISPOSED</td>
<td>Lithology and rock structures</td>
<td>Height, inclination and shape of the slopes;</td>
<td>Pedogenetic structures</td>
<td></td>
<td>. Annual mean temperature: 179°C to 20°C.</td>
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<td>. Rainfall: 1500-3000 mm/year</td>
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<tr>
<td>PREPARATORIES</td>
<td>Erosion-process: &quot;gullies&quot;, ravines, creeping and overhanging of the slopes</td>
<td>Fracturing and galleries on the soils.</td>
<td>Winds predominantly from the South</td>
<td></td>
<td>. deforestation, feeding ground for cattle, type of agriculture, roads.</td>
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<td>EFFECTIVE</td>
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Waves generated by winds coming predominantly from the South acting on the slopes.

**TABLE 1 - CAUSES OF SLOPES INSTABILITY**
Figure 1 is a portion of the final product of the studies: the map of "Homogeneous Sectors of Analysis". This map contains the data listed below.

GEOLOGIC DATA:
lithology, structures and stratigraphy
  PεY Granitoide rocks
  Pεq Quartzites
  PCD Homogeneous stromatic migmatites
  \( \rightarrow \) Transcurent fault
  \( \rightarrow \) Lineament

GEOMORPHOLOGIC DATA:
Landform systems (241) and smaller units (241-A)

\( \rightarrow \) Contact between landform systems
\( \rightarrow \) Contact between smaller landform units

HOMOGENEOUS SECTORS OF ANALYSIS - I, II, ..., XIV

SLIDES TYPES:
- Type A  - Type B  □ Type C  □ Type D

Table 2
PHOTO 1 - "Type a" slide - sliding plane passing through the colluvium. Degradation fractures of the slope: "pied de vache", galleries and fracturing

PHOTO 2 - "Type b" slide - sliding plane developed along residual soil. The steps on the slope correspond to the wave's action according to different levels of the lake.
PHOTO 3 - "Type c" slide - sliding plane passing through foliated rock. The cavern is due to the wave's erosion action.

PHOTO 4 - "Type d" slide - the sliding plane corresponds to granitoide. The wave's action provokes the residual soil desegregation releasing rocky blocks.
PHOTO 5 - Sediment sampled in the bottom of the lake in front of a slide. The left part of the photo shows the silting material while the right one shows the superficial soil before the filling of the lake. The sample was obtained using the "Piston Core".
ABSTRACT

Since the classical case history of seismicity associated with reservoir impoundment of Lake Mead, USA, there have been about eighty instances world over where seismicity of varying intensities was observed following impoundment of reservoirs. A few reservoirs namely: Koyne, Hsinfengkiang, Kremasta, Kariba etc. exhibited intense seismic activity following impoundment, and the maximum earthquake magnitudes were in the range of 6.0 to 7.0. Maximum ground intensities in the vicinity of the dams at Koyne and Hsinfengkiang were in the range of VIII - IX MM scale (0.3 to 0.6g), and these localised high ground intensities could damage the well designed concrete dams at Koyne and Hsinfengkiang. From the well documented case histories world over, the most frequented magnitude range is between 4.0 and 5.0 in the Richter scale. Of the about eighty cases of RIS reported so far, about 55 cases have been accepted as definite RIS while 12 cases are doubtful RIS and the rest are considered not related to RIS. Of the eleven reservoirs where earthquakes of magnitude 5.0 and above occurred following impounding active faulting was present at least in 9 cases, and no fault displacements were observed along the inactive faults. These observations indicate prevalence of triggering phenomena in seismicity associated with reservoir impounding. In some selective case histories, RIS data have also yielded significant information on effectiveness of precursors such as b-values, ground tilt etc. in earthquake prediction.

ABSTRACT

Depuis l'exemple classique de la séismicité associé de la mise en eau du réservoir de Lake Mead, E.U., il y avait environ quatre-vingts exemples dans le monde où la séismicité des intensités variantes a été observée suivant la mise en eau des...
réservoirs. Un certain nombre de réservoirs à savoir: Koyne, Ha infringiang, Kremasta, Kariba etc. ont exhibé l'activité intense sismique suivant la mise en eau et les magnitudes maxima du tremblement de terre de 6.0 à 7.0. Les intensités maxima de terre à proximité des barrages à Koyne et à Ha infringiang étaient dans la portée d'échelle VIII-IX MM (0.3 à 0.6 g), et ces hautes intensités localisées de terre pouvaient endommager les barrages en béton bien étudiés à Koyne et à Ha infringiang. Des exemples bien documentés dans le monde la portée de la magnitude la plus fréquentée est entre 4.0 et 5.0 dans l'échelle de Richter. D'environ quatre - vingt exemples de R.I.S. rapportés jusqu'ici, environ 55 exemples ont été acceptés comme RIS bien déterminés pendant que 12 exemples sont RIS douteux et les autres sont considérés n'ayant pas de rapport à RIS. Des onze réservoirs où sont arrivés les tremblements de terre de la magnitude 5.0 et en dessous suivant la mise en eau, la faille active s'est trouvée dans neuf exemples, et pas de déplacements de la faille ont été observés le long des failles inactives. Ces observations indiquent la prédominance des phénomènes du déclenchement dans la sismicité associée de la mise en eau du réservoir. Dans quelques exemples sélectifs les données de RIS ont rendu le renseignement significatif sur l'efficacité des précurseurs telles que les b-valeurs, l'inclinaison de terre etc. dans la prédiction du tremblement de terre.

Since the classical case history of seismicity associated with reservoir impoundment of Lake Mead (Hoover dam) in U.S.A., there have been about eighty instances worldover where seismicity of varying intensities was observed following impoundment of reservoirs. The prominent case histories of reservoir induced seismicity (RIS) are Koyne, Kremasta, Kariba, Ha infringiang,

Montgomery, Akanfou, Bejina Basta, Benmore, Hoover, Manicouagan 3, Oroville, Volta Grande, Eucuabene, Kinnerami, Kurobe, Marathon etc. Seismicity associated with first four reservoirs namely, Koyne, Kremasta, Kariba and Ha infringiang was moderately intense with maximum ground intensity up to VIII (MM Scale). Modern concrete dams at Koyne and at Ha infringiang were damaged due to locally high ground intensity (0.3 to 0.6 g). The statistics of occurrence of RIS show that the most frequented magnitude range is between 4.0 to 5.0 and this is obviously due to the fact that RIS cases at micro-earthquake level generally escape detection due to its low intensity. Depths of water generally exceed 100 m for maximum earthquake magnitude more than 5.0 in RIS, though some of the deepest and largest reservoirs have not exhibited RIS. In the later category are the giant reservoirs of Bratsk (USSR), Manicouagan 5 (Canada), Aswan (Egypt), Caborra Bassa (Mozambique), Sayansk (USSR) etc. The geographical locations of the RIS cases show that most of the cases including the intensely active reservoirs are situated in moderate to low seismic regions, such as Koyne, Kariba, Ha infringiang etc. So far there have been few cases of RIS in highly seismic and aseismic regions of the world. Implications of these
peculiar distributions of RIS world over have not been fully understood as yet though partial explanation could be advanced from pore pressure hypothesis resulting in reduction of effective stress for origin of RIS.

There have been several instances of seismicity and faulting following petroleum extraction. The reported cases are from USA and Italy. Microseismicity has been observed due to artificial fluid injection along known faults. There have been instances of very local earth movements, even felt, following ground water withdrawal at shallow depths. These above seismic movements seem to have great similarity with RIS connected with large bodies of water though the same have so far received scanty attention. The other commonly observed artificial earth movements are the 'rock bursts' 'bumps', 'outbursts' following mining operations. Some of the rock bursts, specially in deep mining strata, could be recorded by seismographs at distances of few thousands of kilometers and could be disastrous locally inflicting severe damages equivalent to MM scale of VII - VIII. These bursts could reach up to Richter earthquake magnitude scale of 5.0. Close seismic observations have revealed presence of numerous rock bursts of smaller magnitudes in some of the mines, specially deep Gold Mines in South Africa and India. Rock burst intensities and number vary with the mining process and the tectonic set up of the region. All the artificial earth movements, RIS, rock bursts, earth movements following fluid extraction and injection seem to depend predominantly on ambient tectonic set up and artificial stress field, both its extent and magnitude. The present paper will, however, broadly deal with reservoir induced seismicity (RIS).

In general, there had been fewer damages to dams due to earthquakes in the past. The main hazards to dam from earthquakes are due to surface faulting, strong ground movements, water waves in reservoir induced by seismic waves or by landslide and rock falls and ground deformation associated with faulting. The ground deformations have been observed in number of cases such as the Baldwin Hills reservoir, Buena Vista Hills, Kern County, at locations over the San Andreas fault etc.. A number of dams was damaged during Feb.9, 1971 San Fernando earthquake, and the significant ones are the upper and lower San Fernando dams in which severe cracking and settlement developed. The Pacoima arch dam however sustained severe ground motions with comparatively minor damages. Cases of damages to other earth dams are Dry Canyon dam (1952 Kern County earthquake), Sheffield dam (1925 Santa Barbara earthquake), Hebgen dam (1959 Montana earthquake) and the Eklutna dam (1964 great Alaska earthquake). Of these, the Eklutna dam was very severely damaged and was later reconstructed. The RIS on the other hand poses hazard to dams and pertinent structure through unforeseen rapid and intense movements originating close to the dam.

Seismic instrumentation to dams and
reservoirs are planned to assess the level of preimpounding seismicity, to follow up the progress of observed seismicity and finally to record seismic forces (accelerations) experienced on the dam and on the ground in the event of a sizeable earthquake occurring in the area. Pre-impounding seismicity being generally of low level, closely spaced dense seismic nets of five or more stations are need to record the micro-earthquakes. The micro-seismographs generally used for the purpose are of high gain ($\sim 10^5$) having frequency response up to 80 Hz. Linear size of such sensitive seismic nets could be around 30 to 50 km. Dams specially higher than 100 m should normally be instrumented with accelerographs to record earthquake forces in the event of occurrence of large earthquakes in the close vicinity of the structure.

Case histories of few important RIS are described below:

(a) Hainfengkiang dam, China:

The concrete dam is 80 m high and is located 160 km north-east of Canton. Impoundment began in 1959 and an earthquake of magnitude 6.2 occurred in 1962 following rapid rise of lake level Fig.1. In subsequent years, a number of earthquakes of magnitude more than 3.8 occurred in the lake area. The earthquakes were over had shallow focal depths indicating a typical RIS case (Oiice et al, 1981). Within a period of 28 months, a total of eighty thousand earthquakes was recorded. Isoseismals of the main seismic event (M = 6.2) are shown in Fig.2. The seismicity, not only induced following impoundment, was of shallow depth (5 to 7 km) and concentrated near the lake area and exhibited variations with changes in lake level. Decay of seismic activity was very slow. Hainfengkiang like Kojia is a typical RIS case and has great parallelism with seismic activities at Kojia. The similarity in geohydrological and tectonic environment could be the determining factors in the two cases.

(b) Kremasta and Marathon dams, Greece:

An earthquake of magnitude 6.3 occurred in 1966 soon after impoundment. The lake area is situated in tectonically active area. Impoundment could have hastened the occurrence of the earthquake. This is a clear case of triggering following impoundment.

(c) Kariba dam, Rhodesia/Zambia:

Kariba lake area, one of the largest artificial water mass in the world, was the seat of series of earthquake of magnitude 5.7, 6.1, 5.6, 5.8, 5.5 and 6.0, vide Fig.3, following impoundment in 1960. The first significant earthquake $M = 5.7$, took place in 1962. Presently, the activity is on the decline though at a very slow rate. The seismicity pattern at Kariba differs significantly from that at Kojia and Hainfengkiang.

(d) Nurek dam, USSR:

The dam is situated in seismic region of Tadikistan of central Asia and has been designed to withstand an earthquake of magnitude 6.5. The lake area experienced an earthquake of magnitude
4.5 in 1972 following rapid increase in lake level.

(e) Oroville Dam, USA:

The dam site has been known to be a low seismic area, and the Oroville earthquake of magnitude 5.7 in 1975 with epicentre near the reservoir has been one of the recent instances of RIS. Low b-values and microearthquake activity prior to the main earthquake near the reservoir are the other characteristics of RIS.

(f) Koyna dam, India:

Following impoundment in 1962, microearthquake activity was recorded, and till 1980, about 50,000 earthquakes were recorded. The main earthquake of magnitude 7.0 in Dec., 1967 was preceded by low b-values, large ground tilt and maximum lake level and since then, seismic activity has been on the decline. Though at a very slow rate Figs. 4 and 5. There has been about two dozens of earthquakes of magnitude 4.0 and above during last two decades of activity at Koyna. Low b-values were observed before both significant earthquakes of magnitude 7.0, Dec. 1967 and 5.2 Oct., 1973. b-value has been a very powerful presonitory index for prediction of RIS, specially the large significant earthquakes, Gaba et al., (1981). Similar decrease in b-value has been observed in case of Mula (India), Itukki (India), Manicouagan 3 (Canada) and Kariba (Zambia/Rhodesia) reservoirs.

Discussions and Conclusions:

Fig. 6 shows the reported RIS cases worldover including the recent seven case histories from China. Figs. 7, 8 and 9 give relative graphical relationship amongst number of RIS reservoirs, 'time lag' between start of impoundment, first RIS events and largest RIS events and magnitude of the largest RIS events. These statistical data along with hydro-geological and geotectonic informations form the data base of assessment of potentiality for any geologic environment for exhibiting RIS (Packer et al 1979).

Fig. 10 shows the histogram of the reported cases, and the most probable magnitude range is between 4.0 and 5.0 in RIS. It is very likely that with increasing trend of instrumentation of reservoirs, more and more case histories with low level of seismicity will be reported. The reported RIS cases have been divided into three categories:

III (M ≤ 3.0) mild, II (3.0 ≤ M ≤ 5.0) medium, I (M ≥ 6.0) intense. Intense RIS activities have been reported from four reservoirs namely Koyna, Kremasta, Kariba and Hsinfengkiang.

The parameter b-value has been found to be effective as a presonitory index in prediction of maximum earthquake of RIS, such as in Koyna, Kariba, Itukki, Mula and Manicouagan 3. In Koyna and Kariba, b-value variations could be used effectively in prediction of variations of seismicity — vide Figs. 11 and 3. Intense seismicity in Koyna in 1967, 1973 and 1980 followed low b-values, and similar has been the case with Kariba and Mula (India). The other presonitory indices observed are large ground tilt observed in Koyna, Mula and Itukki and
magnetic anomaly and underground rock strains. Some of these premonitory indices specially b-value variations show premonitory period \( T = 0.76 M^{-1.83} \) for the maximum earthquake in HYS. These observations directly confirm that the broad mechanism of variations of ambient stress field following impoundment is similar to normal geotectonic process in earthquake though the superimposed water load initiates triggering process for seismo-genic movements. In view of close range observations of premonitory indices such as b-values, tilt, strain, magnetic field etc. possible use of these above parameters in short term prediction could be studied. Fig. 12 shows the large accelerated tilt at Koyna dam shortly before the maximum earthquake of magnitude 7.0 indicating the possible use of ground tilt close to epicentre as a short term premonitory index. It is very much possible that other premonitory indices could also exhibit similar short term prediction capability.

It has been observed in Koyna, Sinfengkiang, Idakki, Narek etc. that both HYS intensities and lake level did reach maximum almost simultaneously, and this observation strongly suggests triggering effect of lake level in inducing HYS. For the Chinese case histories of HYS in Fig.13, magnitude of maximum earthquake is directly proportional to depth of water column in the lakes while Simpson (1976) has shown Fig.14 that neither depth of water column in lake nor reservoir capacity could adequately explain the magnitude of maximum earthquake in HYS. According to Oike et al. (1981) Simpson's results could be due to the influences of divergent tectonic set up of the reservoirs. Moreover, limited studies at Koyna and Mula have confirmed the very significant correlation of rate of change of lake level \( (dL/dt) \) with occurrence of maximum magnitude earthquake in HYS, though it is necessary to investigate the above correlation in other HYS cases. Rowles (1974) on the other hand has shown, Fig.15, that time required for significant pore pressure to migrate at depths 5 to 7 km could be of the several hundred days which is corroborated from observations of 'Time Lag' between commencement of impoundment and occurrence of maximum earthquakes such as in Koyna, Kariba, Montepourd, Manicouagan 3, Narek etc.. These observations of HYS and theoretical deductions on pore pressure migration in rock lend credence to the effectiveness of pore pressure in initiating seismogenic movements along existing fault planes in moderate to low tectonic environment though large rate of change of lake level \( (dL/dt) \) could perhaps trigger the onset of maximum magnitude earthquake as evident from Fig.11. Thus fluid pressure has thus dual role in HYS, proper manipulation of lake level \( (L) \) and its rate of change \( (dL/dt) \) could be effective in controlling the resulting HYS, specially the occurrence of the maximum earthquake magnitude. If rate of change of lake level \( (dL/dt) \) could be maintained below the threshold value, HYS could be dissipated amongst larger number of smaller
smalls spread over longer time. Though
limited observations at Koyana have con-
formed the above hypothesis, the same
required more observational data spe-
cially in diverse tectonic environment
for further confirmation. Fault plane
mechanism study has shown Simpson (1976)
that normal or strike slip faulting
favour RIS, Fig.16. Thus judicious
study of tectonic and geohydrological
environment and hydraulic input of the
reservoir could be helpful in controlling
RIS. The artificial experiments at
Banglay oil field have shown that
seismicity depends on the rate of
pumping and once triggered, it may
continue for sometime.

The principal mechanisms of RIS
which seem feasible could be:

(i) the reservoir load generated
stress, provide the 'last straw'
for the seismogenic movements.

(ii) the increase in pore pressure
following impoundment reduces the
effective stress necessary for
crustal readjustments.

(iii) reservoir load itself could cause
the earth movements.

Reservoir load being small (~ 10 bars),
the pore pressure could alter the
effective stress thus inducing seismo-
genetic movements. The influence of depth
of water column on the resulting RIS
magnitude, Fig.13 and the corroboration
of observed 'Time Lag' in RIS from pore
pressure migration hypothesis, Fig.15
lend credence to the pore pressure
hypothesis in initiating RIS though the
exact mechanism remains somewhat ambigu-
ous. Quantitatively, the maximum earth-
quake magnitude ($M$) in RIS, could be
equivalent to effective stress drop ($P$)
given by

$$P_0 - P_1$$

(tectonic stress) (pore pressure)

Thus data on geotectonics, geohydrology
and reservoir hydraulics could be utilised to anticipate and perhaps
control RIS in favourable conditions.

In case of intensely seismic areas
like the Himalayas where $P_0$ is large, hence no seismogenic movements
could be possible while in aseismic
regions with small $P_0$, the resulting $P$
so also earthquake magnitude would be
small. In aseismic areas like interior
of shield regions, low level micro-
seismicity (RIS) is generally observed.
Hence only in moderately seismic areas,
significant RIS could be possible.
Geographical distributions of RIS, Fig.6
confirm the above deductions of RIS.
Hydro fracturing experiments could give
vital data for assessing potentiality
of RIS at any geologic environment.
Many of these conclusions and deductions
may be equally applicable for artificially
induced seismogenic movements such as in
RIS, seismicity following fluid extraction
(petroleum), withdrawal (ground
water) and also injection and 'coal bump',
'rock bursts' etc.

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Fig. 1. Lake level and seismic activity at Hsinfengkiang dam (Oike et al., 1981).

Fig. 2. Isoseismals of the main earthquake of March 19, 1962 at Hsinfengkiang dam (Oike et al., 1981).
Fig. 3. Seismicity, b-values and lake levels, Kariba reservoir.

Fig. 4. Significant Koyna earthquakes, b-values, ground tilt, earthquake fracture volume and lake levels.
Fig. 5. Isoseismals IX to V of the main earthquake of December 10, 1967 at Knysna dam.

Fig. 6. Reported RIS cases of the world.
Fig. 7. "Time Lag" between impoundment and first RIS event (Packer et al, 1979).

Fig. 8. "Time Lag" between impoundment and larger RIS event (Packer et al, 1979).
Fig. 9. Magnitude of the largest RIS event versus "Time Lag" of accepted cases of RIS (Packer et al., 1979).

Fig. 10. Magnitudewise distribution of reported RIS cases.

Fig. 11. Significant Koyna earthquakes, b-values, deflection, strain and rate of change of lake level.

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Fig. 12. Ground tilt and significant earthquakes at Koyna reservoir.

Fig. 13. Magnitude of induced earthquakes and dam heights of RIS in China (Oike et al., 1981).

Fig. 14. Magnitude of largest induced earthquakes, reservoir volume and dam height (Simpson, 1976).
Fig. 15. Increase of pore pressure at various depths (Howells, 1974).

Fig. 16. Fault plane solutions for earthquakes at reservoirs with major induced seismicity (Simpson, 1976).
A NOTE ON RESERVOIR INDUCED SEISMICITY

UNE NOTE SUR RESERVOIR DE SISMICITE INDUITE

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ABSTRACT

A number of cases are reported over the last two decades or so where reservoir filling is considered responsible for the observed seismic activity in respective areas. Not all high dams or large reservoirs, even under similar geological environment, exhibit associated activity. The data and statistics of these negative cases is generally not studied. Also even amongst the positive cases the timing of so-called triggered seismic activity have ranged from the starting of reservoir filling to the time of their being filled to the maximum capacity. Number of facts in each case cannot be explained coherently by any single physical model which may be proposed to support the role of reservoir filling in inducing seismic activity. A discussion on the subject is presented.

To permit an objective conclusion about the possible interrelationship of reservoir impounding and seismic activity, uniform quality seismic data both for pre-and post-impounding conditions need to be collected for some future sites of large reservoirs. Studies initiated well in advance of reservoir filling by the Department of Earthquake Engineering, University of Roorkee on this problem at two sites have been presented in the paper.

ABSTRAIT

De nombreux rapports sont parvenus pendant les deux dernier décades où l'on a signalé que le remplissage du réservoir a provoqué une activité sismique observée dans une région respective. Tous les grands réservoirs on les barrages élevés, même sous la condition géologique pareille, n'ont pas une telle activité. Les données et des statistiques de
ces cas négatifs ne sont généralement pas étudiés. Même chez les cas positifs, le temps d'activité seismique de détente est porté des le remplissage des réservoir jusqu'à la capacité maximum. Les nombres de faits en chaque cas ne peut pas expliquer cohérentement par le seul modèle physique proposé pour supporter le rôle de remplissage des réservoir d'introduire l'activité seismique. Une discussion sur le sujet est présentée.

Pour permettre une conclusion objective sur l'interrelation possible entre le réservoir enfermer et l'activité seismique, les données seismiques de la qualité uniforme pour toutes les deux conditions enfermer - avant et après - doivent être rassemblées pour les grand réservoir postérieurs. Les études initiées bien avant des remplissage du réservoir par le Department of Earthquake Engineering, University of Aorkee sur ce sujet dans deux régions sont présentées dans ce travail.

**INTRODUCTION**

The number of reservoirs with greater than 100m tall dams more than doubled in the world during 1960's. This trend of increase in the number of tall dams has been continued over last two decades. Also, as the good dam sites are being exhausted the newer ones are being built in less favourable sites with even high seismicity. Over the last two decades the number of cases where the reservoir impounding has been considered responsible for the observed seismic activity in respective areas has also grown (Simpson, 1976). It is of scientific interest to note that not all high dams and large reservoirs even those under similar geological environment exhibit seismic activity. If the statistics of negative cases is compiled the data on so called cases of induced seismicity becomes statistically insignificant. The positive correlation between the activity and reservoir impounding if any observed may be reflective of the causative relationship or may be merely a matter of simple chance. Unfortunately most of the studies on the subject attempt to substantiate the hypothesis of induced seismicity rather than to collect and analyse scientific evidences to arrive at a conclusion with an open mind.

The physical conditions that are considered essential to enable causative relationship between the reservoir impounding and seismic activity can be summarized as pre-stressing of rocks, presence of rock defects and porous but low permeability rocks (Kisslinger, 1976). Even with all the condition being satisfied the number of negative cases is alarming. Also the induced seismic activity in some cases accompanied with starting of filling
whereas in some others it coincided with the filling to the maximum capacity. Perhaps no simple physical model could explain this sort of feature of the timing of induced activity in a consistent manner.

A review of the past cases of so-called induced seismic activity further reveal that in most cases data on seismic activity prior to impounding was not available. Even in those cases where some data was available during the pre-impounding stage, the quality of the same has been generally dissimilar to that obtained during or in post impounding stage, not allowing a uniform analysis and fair comparison. It is with this background that studies have been initiated well in advance of reservoir filling at two sites in India by the Department of Earthquake Engineering, University of Roorkee. These
sites falling under very dissimilar geotectonic setting are Navagam and Tehri Dam regions. Some of the preliminary results of the survey in these regions are presented in this paper and generally highlight the need for more intensive instrumentation for the purpose to be able to get objective conclusions.

NAVAGAM DAM

A 142m tall concrete gravity dam is to be constructed on river Narmada in Gujarat State, India. The region falls close to the Sone-Narmada lineament, Piplod fault and other tectonic feature and is located in Deccan Traps. The reservoir will have a very large linear extension (about 250 km) and does not exceed 4 km in width which generally is only about 2 km or so. A short term (six months) five station microearthquake recording has been conducted during 1980 (Report Eq 81-9, 1981) to sample the typical seismic activity much in advance of the dam construction and reservoir impounding.

The locations of recording are shown in Figure 1. Initially the aperture of the array was very small (site Nos. 2 to 6 in Fig. 1). Subsequently the aperture was expanded. The locations chosen at the expanded array were such that some of those would remain available for recording after the reservoir impounding to enable comparative study. Changes in recording sites were also done to improve simultaneous recording of events and also to take care of the problems due to the onset of monsoon. In total nine sites were occupied as shown in figure 1 and for a period of 25 days six station simultaneous recording was also possible.

A total of 310 events within 125km distance were located of which 108 were of magnitude 1 or more. Out of these, 233 events are plotted in the map in figure 1. The activity is by and large scattered and does not show definite relationship to the tectonic features mapped in the region. However, if the activity below zero magnitude was taken away, some epicentres could be seen aligning themselves along the Sone-Narmada lineament and Piplod fault both running roughly NE - SW. In addition to the locations aligning along these features, the similarities of appearance of their traces and comparability of estimated focal depths were suggestive of their possible association with respective features. The recorded activity is also suggestive of the possible extension of these features to further SW. Another sample of seismic activity through a 2 months tripartite microearthquake recording during 1976 (Final Report Eq 78-2, 1978) has shown activity along Raj Parah fault which was not detected during 1980 survey.
FIG. 2 RECORDING SITES AND EPICENTRES LOCATED IN TEHRI DAM REGION

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TEHRI DAM

A 260.5m tall earth and rock-fill dam, tallest of its type in India, is to be constructed on river Bhagirathi in Uttar Pradesh. The region falls in the lesser Garhwal Himalayas between the Main Boundary Fault (MBF) and the Main Central Thrust (MCT). Numerous other tectonic features have also been mapped in the region. The shape of the reservoir is in the form of a fork extending into the valleys of Bhagirathi and Bhilangana rivers and would cause submergence of the local tectonic features presently exposed on the surface. A short term (110 days) tripartite microearthquake recording has been conducted during 1974 to 1977 at various random time intervals.

The locations of recording are shown in Figure 2. Initially the recording was also done at Tehri but finding the activity mostly to further north of the area the same was moved towards north. In all five sites of recording were occupied (Kumar, et al., 1980).

A total of 115 events within 125km distance were located. Most of the activity recorded is between magnitude 0 and 2. Out of the located events only 88 are shown in Figure 2. The activity in the vicinity of MBF seems to be less compared to that close to MCT. The region close to Uttarkashi which is conspicuous on a tectonic map due to presence of numerous thrusts and faults is also showing clustering of epicentres. The association of epicentres with the local tectonic features could be seen clearly. In another study for the same region (Report EQ 79-16, 1979) the data from Nandna Nagar, Tehri and Haridraprayag observatories for about 3 years was used to locate events which were dominantly of magnitude 2 to 3. The epicentral map for the activity thus located showed greater scatter and did not permit any conclusion about association of local tectonic features and the recorded activity.

DISCUSSION AND CONCLUSIONS

The short term sampling of seismic activity in the region of Navagam Dam during 1976 and 1980 showed substantially different picture and is a clear evidence of temporal changes in activity. Compared to Tehri region the activity is generally low if the sizes of events were diagnostic parameter. In Navagam Dam region a plot of epicentres after filtering of low magnitude events improved the identification of possible interrelationship between activity and tectonic features. In case of Tehri dam region it was just the opposite. The contrast in the characteristics of the seismic activity recorded during short term surveys are clearly related to differences in the tectonics of the two regions. Perhaps, the low level seismic activity in the
Navagam Dam region is due to some surficial cause and not associated with the tectonic features mapped there. Whereas the low level activity in Tehri region was clearly associated with tear faults and relatively bigger events were scattered along the regional features.

In case substantial variation in seismic activity are demonstrated even in the absence of reservoir impounding it will be a very difficult task to establish which variations are attributable to reservoir impounding. More intensive instrumentation on permanent or semi-permanent basis is required for such an objective. Suitable recommendations for the desired instrument have been made to the respective agencies. Hopefully a timely installation of suitable seismic network of stations in these two sites will enable better understanding of the phenomenon of induced seismic activity.

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ENGINEERING GEOLOGICAL ANALYSIS METHOD FOR THE FORECAST OF RESERVOIR INDUCED SEISMICITY

LA METHODE DES ANALYSES GEOLOGIQUES POUR LA PREVISION DE RESERVOIR DE SISMICITE INDOUITE

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ABSTRACT

The causes of formation of reservoir induced seismicity were first discussed in this paper. The temperature stress produced by the interaction between the reservoir water and the deep rock mass, both dry and hot, was proposed to be the induced force.

On the basis of discussing the causes of formation, a method of engineering geological analysis for the forecast of reservoir induced seismicity was put forward. It is considered that the occurrence of reservoir induced seismicity must have three conditions simultaneously, i.e. a large quantity of water seeping through the reservoir base down to a certain extent of the rock, a deep tectonic condition that can create a considerable water pressure difference, and the existence of certain rock property. Thus, it is possible, in combination with the geological exploration, to forecast the occurrence of the earthquake before the construction of the reservoir. Consequently such a method seems to be a realistic and feasible one at the present time.

ABSTRAIT

Tout d'abord, la formation du seisme dû au réservoir a été discutée dans notre article. Nous avons indiqué une notion suivante, que la contrainte de temperature, introduite par l'action réciproque entre l'eau dans la réservoir et le massif rocheux sec et chaud, c'est une ressource en force inducetice de seisme du au réservoir.

Sur cette base, une méthode de prévision du seisme dû au reservoir par l'analyse de la géologie d'ingenieur a été presentée. On admit que, le mise en action du seisme dû au réservoir doit soumettre à trois conditions principales à savoir.

— le grand volume d'eau, infiltré dans un certain rayon dessous le reservoir
— les conditions tectoniques profondes, favorables pour introduire une grande chut de potentiel hydraulique
— les caracteristiques de massif rocheux.

Alors, la prévision du seisme du au réservoir avant sa realisation sera possible, si les études de la géologie d'ingenieur dans la zone d'influence du réservoir a été fait attentivement.

Il semble que, cette méthode soit assez pratique et effective pour être d'une application courante.
INTRODUCTION

Before constructing a reservoir in a region, it is of great significance to forecast whether there is induced seismicity after the reservoir is completed. It not only has an important bearing on the design, construction and utility of a hydraulic project, but directly influences the safety of the cities, factories, and mines, human lives and properties near the reservoir.

Ours is a country where earthquakes occur frequently, and we already found there are until now 13 induced seismicity reservoirs, about one sixth of the total seismicity reservoirs in the world. From now on, as the development of our national economy proceeds, more reservoirs will be constructed. Thus, there is an urgent requirement before planning a hydraulic project, to predict whether there is induced seismicity after the filling of the reservoir.

Although, the foci of the reservoir seismicity is rather shallow, yet we still are unable to measure and understand the progress and mechanism of the seismicity exactly. Under this situation, it is difficult and unrealistic to forecast precisely the reservoir induced seismicity. At the same time, the prediction of the reservoir seismicity is not like that of the tectonic earthquake. We may utilize the progress of the earthquake prelude to the development of earthquake. In order to meet the necessity of hydraulic construction, it is possible to predict qualitatively the reservoir induced seismicity and to provide the basic considerations for engineering design before the question of exact forecast is solved, based upon the geological conditions of the reservoir area, and with the aid of engineering geological analysis method. In this paper to do some works on this line is attempted.

CAUSES OF THE RESERVOIR INDUCED SEISMICITY

With regard to the causes of the reservoir seismicity, many investigations and discussions have been made, but the problem is still not satisfactorily solved, many different points of view are existing, and so there must be many different ways and methods to predict the reservoir earthquake.

Reservoir earthquake is a man made one. The construction effect of the reservoir water induces the earthquake. As to the origin of the seismicity, it is not entirely similar to the tectonic earthquake-tectonic movement origin. Therefore, there are two different kinds of earthquake. Without reservoir, there will be no reservoir earthquake.

Regarding the causes of the reservoir seismicity, it has been proposed that it is caused by the effect of load of the reservoir water, and by the effect of the seepage of the reservoir water, etc. There is still some arguments on the effect of load of the reservoir water. Of course the weight of water in reservoir is tremendous. The fact that in the basin of earthed quake reservoir appears vertical and horizontal movement support this point of view, but it is still uncertain that it induces earthquake. We considered this as the result of the tremendous water weight, simultaneously, there may be the geotectonic movement due to the process of the earthquake.

The seepage theory is generally accepted. This theory implies two different meanings, one is that the fault area is under the critical condition of the fracture disturbance, i.e. under condition of high tectonic stress. After the reservoir water seeps into faults, it lowers the strength of the fault, simultaneously, due to the increase of the water pressure on the fault, the principal stress of resistance of fault disturbance is decreased, in turn it renews the disturbance of the fault and may induce earthquake. The other meaning emphasizes that the major effect of reservoir water seepage through the rock mass has created a great difference in seepage pressure. In other words, the water seeps through the passage toward the deeper part. As the seepage goes deeper, the water pressure becomes greater, at the area that reservoir seepage occurs a high seepage pressure zone is formed and pressure difference, exists between the impervious rock mass and the seepage area. As the pressure difference exceeds the rock strength, the rock mass fracture disturbance will occur.

Besides, the author realizes that water and geothermal produce temperature stress, this is also a source of power to induce reservoir seismicity.

It is well known that within the earth contains tremendous heat energy. Below the crust the geothermal temperature rises as the depth increases. This also is held true in the reservoir area. The depth of the foci of the reservoir seismicity is usually within 20 km. At this depth the temperature of rock mass may reach several hundred degree centigrade. In case of extraordinary high geothermal
area, the temperature is even higher. It can be imagined, therefore, when reservoir seismicity origin is under the dry-hot condition, the temperature of reservoir water is comparatively low. If great quantity of water through the fault to the rock located deep below, and meets the dry-hot rock, two kinds of reaction will happen, one is to heat the reservoir water, raising temperature, swelling volume and aeration, as well as accumulating energy, to quicken the rock fracture. On the other hand, the dry-hot rock meets reservoir water with comparative low temperature, due to the sudden drop of rock temperature, the fracture surface sudden shrinkage, produced stress, and large amount of rock surface small cracks are formed. If this action takes place incessantly, cracks will continue to open up as the high seepage pressure goes through. Consequently introduces the stress concentration, causes large cracks and dislocation of rock mass, even breaks up earthquake. At the beginning, the elementary rock cracks require not much energy, a small fracture is enough to introduce stress concentration. Therefore the action of the reservoir water and dry-hot rock mass is possible to induce earthquake. In our country, a lot of the reservoir regions associated earthquake all appeared with hot springs. This shows the earth temperature is relatively high. During earthquake, warm water blows out and high temperature aeration, these phenomena are related to the above mentioned actions.

It can be seen that the effect of reservoir water induce seismicity in many respects, except the effect of fluid pressure on the fault. The thermal stress induced by water and dry-hot rock mass interaction is also one of the power source of the reservoir seismicity.

PREDICTION METHOD OF THE RESERVOIR SEISMICITY

This paper proposed a prediction method of the reservoir seismicity, a kind of engineering geological analysis method. Based upon the causes of the reservoir seismicity, there are only three conditions for determining whether reservoir construction may induce seismicity: the seepage condition of the reservoir area; deep tectonic condition, and the rock medium condition. If all the three conditions are provided at the same time, it can be assured that the reservoir will induce seismicity. If only provided one or two conditions, the reservoir will not induce seismicity. In this case, with the aid of engineering geological survey, it is possible to predict the reservoir seismicity before the design and construction work are carried out. Now we are going to discuss the three conditions in more detail as follows:

1. Seepage Condition of the Reservoir

Seepage condition of the reservoir is the most important item in induced reservoir seismicity. According to the causes of reservoir seismicity, no matter what it is, seepage pressure; temperature stress; as well as to lower the friction resistance of the fault surface, they are produced only since the water of reservoir seeps into the rock mass. It may be recognized, if no large amount of reservoir water seeps into the deep rock mass, there will be no induced power origin produced, also no induced reservoir seismicity.

With regard to reservoir seepage condition, it is necessary to satisfy the three requirements:

1) Existence of Seepage Passage in the Reservoir Deep Rock Mass

There are many reservoir areas, though provided seepage condition in the reservoir basin face layer, water may seep down into the earth crust for tens of meter, then it may be stopped by imperious layer. In this case, the water cannot seeps deep enough to the earth crust, there will not be enough water to induce seismicity. For example, in Liehui Reservoir, Hubei, some areas of the reservoir consist of limestone, with many holes and is a good passage of seepage. However, below the limestone layer there is thick shale layer, which stops the seepage at its top layer. This is the main cause why this reservoir has no induced seismicity.

In analysis of the seepage condition, it should be included that during the maximum water level of the reservoir whether there are many seepage passage in the flooded area. In this area, before constructing the reservoir, it is easier to find out the seepage passage and take counter measures. If the maximum water level of the reservoir is lower than the historic flooded marks, it is better to check the earthquake record during the maximum flooded period in order to determine the possibility of the reservoir seismicity.

2) Seepage Area must be Large Enough

If within the reservoir area, only in a very small portion exists seepage condi-
tions, then the effect of the reservoir seepage is not great. Therefore, from the view of the induced seismicity, the reservoir seepage area and scope should be provided considerable scale, otherwise these will be not good enough for inducing seismicity.

(3) Great Quantity of water Seeps down into Deep Rock Mass

In the reservoir of the induced seismicity, the amount of seepage water to depth is considerable, roughly estimated to be more than several million cubic meters. It is not difficult to understand, if the reservoir water seeps several kilometers deep, and extends to several kilometers wide, the necessary total amount of water to fill the rock cracks is considerably large.

2. Deep Tectonic Condition

From the view of induced seismicity, presents the following two important items to analyse the deep tectonic condition in the reservoir area:

(1) Existence of Tectonic Environment which may create a Difference of Water Pressure Environment.

Even if there is large amount of reservoir water seepage into deep ground, when within the rock there is no possibility to build up a large water pressure difference, it is not easy to break the rock and inducing seismicity. When the water seepage reached to the deep cracks and plugged up not to expand the seepage area, then the water in the deep cracks will hold certain high pressure. This pressure is produced from the seepage, and at the nearly impervious rock mass as no seepage water existence, hence there is no seepage pressure. Therefore between the two different pervious rock masses, there will occur pressure difference. If one side of the fault is pervious or contact of two different perviousness of rocks, they all create the tectonic environment of the water pressure difference.

(2) Existence of Fault and Weaken Zone

Although the existence of fault and weak ground is not the cause of earthquake, yet its existence greatly decrease the rock strength. Such that it is easier to concentrate the stress and produce earthquake. Reservoir seismicity may occur along the fault zone, also this may be due to the continuity of expansion of cracks in the weak ground.

3. Rock Medium Condition

The outbreak of seismicity is closely related to the rock medium. The mechanism of fracture of the plastic and that of the brittle rock are quite different, and the style of stress release also not the same. When the plastic rock is under load and deformation, there will be a very large displacement, but no vibration produced. On the contrary, the fracture of brittle rock is followed by vibration. Therefore most reservoir seismicities occur in limestone, granite, basalt, and gneiss, etc. hard and brittle rock district. On the midstone, shale and thick newly sedimentary overburden area construct reservoir, generally there will be no induced seismicity.

The other item of the rock medium is the different of rock characteristics, include the difference between horizontal and vertical directions. Difference in rock interface properties, the heave of the interface, they are not only different in perviousness, may form water pressure difference, simultaneously, may form highly concentrated stress, and along the interface produces fracture movement.

CONCLUSION

Forecast reservoir induced seismicity is a relatively difficult and complex problem. In order to solve this problem, it is necessary to find out the causes of the reservoir seismicity, clearly understand its mechanism and master the corresponding survey technique. Such as determine the possible location of fracture, strength and direction, define the original earth stress field, determine the newly produced stresses after filling the reservoir, after the adjustment with the original earth stress field whether exceeds the strength of the fault surface, as well as the heave and development of the fault surface, etc. Now all these problems are still not fully solved, and to make quantitative prediction. Likewise the sequence of the reservoir seismicity is progressing, then the problem is more complex. Before trying to solve these problems, application of the approach proposed in this paper, by using the three conditions to make engineering geological analysis. It will be possible to evaluate the possibility of the induced seismicity before the reservoir is constructed. Therefore the proposed method can be considered as a comparatively realistic one.
REFERENCES


THE TECTONIC ENVIRONMENT OF THE INDUCED EARTHQUAKES IN HSINFENGCHIANG RESERVOIR

L'ENVIRONNEMENT TECTONIQUE DU TREMBLEMENT DE TERRE INDUIT PAR LE RESERVOIR DE HSINFENGCHIANG

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ABSTRACT

The induced earthquakes associated with the impoundment of a reservoir concentrate as a rule at the margins of the lake and may be divided into two layers according to the focal depth. Between them there is a rare earthquake layer. In Hainfengchian Reservoir there are distinct differences between the faults which caused the shallow earthquakes and those which produced the events at deeper levels. But all of them obviously are active faults with high dip angles, which play a much more important role than those with low dip angles in the formation of the induced earthquakes.

ABSTRAIT

L'origine et la distribution de tremblement de terre induit causé par l'accumulation du réservoir ne seulement décidé de l'environnement tectonique local, mais il y a des étroites relations avec la situation de pression additionnel provoqué par la charge de réservoir. C'est pourquoi, le tremblement de terre induit a concentré en général au bord du reservoir. On peut le diviser en deux étales selon les différentes profondeurs de foyer séismique. Parmi les deux étages, il y a un rare-étage seismique. A Hainfengchian, il y a des manifestes différences entre la tectonique de séisme induit à étage peu profond et à étage profond. Mais les tectoniques de séisme induit à étage peu profond et à étage profond sont visibles dislocations actives en haut-coin. La dislocation de haut-coin est joué un rôle plus important que celle de bas-coin dans le période de séisme induit.

1 SEISMOGENIC STRUCTURES

The Hainfengchian Reservoir is located mainly on a Cretaceous granite batholith intruded into a series of sedimentary rocks of the Upper Paleozoic and Mesozoic era. To the east of the Hainfengchian Dam, it is linked with a fault basin of the Cretaceous-Paleogene period (Fig. 1). In this area, structural faults are well developed. There are three main fault series, whose strikes are respectively N20°-40°E, N15°-35°W and N70°-80°E.

The thrusts and overthrusts with NNE strike are the most developed surface faults in this area. Nevertheless they have no direct relations with earthquakes.

The faults of NNW strike with an inclination of ENE or WSW, the principal dip angle being 70°-90°, have mainly developed on the east part of the lake. This series of faults is much lesser than the NNE strike series, both in scale and in the amplitude of dislocation. Roughly speaking, they are lateral faults ranging from several meters to few kilometers in
length. Sometimes they may also take the form of intensive shear joints. Their sliding track is left-lateral motion. The seismogenic structures of earthquakes including the main shocks and some of the stronger after-shocks that occurred in this area all belong to this series of faults. Seismic and geological research asserts that the faults of NWV strike are active ones, as they not only intersect and displace the NNE faults, but also have had relatively more obvious activity since the Quaternary period. During the period of more than two decades before the reservoir began to store water (1937-1959), the three sensible earthquakes of this area all took place in the vicinity of this series of faults (Fig. 2).

The faults of ENE strike are located at the northern and southern sides of the reservoir and in the vicinity of the reservoir canyon. They are nearly vertical, and their motion is mainly right-lateral, in some cases dip slip are the principal form this motion takes.

The aftershock distribution and focal mechanisms demonstrate that both kinds of fault are the seismogenic structures of some of the aftershocks. This series is not developed on the earth's surface, although electric resistance evaluation and exploration research indicate it is the most important structure of the basin of the Cretaceous-Paleogenin basin which is divided by it into two parts. The AT plane figure made by aero magnetic measurement (Fig. 3a) and gravity anomaly plan of this area (Fig. 3b) demonstrate that the ENE faults are the main structure in the deeper part of earth's crust. The residual gravity anomaly obtained by continuing to a depth of 10 km and eliminating the impact of gravity anomaly near the surface indicates that this phenomenon with regard to the ENE faults is even more evident.

In short, there are two series of faults in this area of the Hsinfengchian Reservoir, their strikes being NWV and ENE respectively. They are both nearly verti-
1. Isoseismal line of main shock; 2. seismic region of M=4.3 (May 4, 1981); 3. the epicentral location of three sensible earthquakes before impoundment of reservoir; 4. fault; 5. deduced fault.

**2. MATHEMATICAL SIMULATION**

Let's exert external forces on a given geological model by the finite element method and try to reconstruct the past seismic events in order to find out how the external forces acted to cause the earthquake sequence, to determine the stability of various series of faults and to give the coefficients related to the seismic events.

After calculation and comparison, the following simplified model is adopted: choose a granite slate at the depth of 5 km beneath the earth's surface, 1 km thick, 69 km long (E-W) and 56 km wide (N-S), insert the seismic-concentrated
c. AT plane figure of aero-magnetic measurement; b. gravity anomaly plan of this area; c. the residual gravity anomaly obtained by continuing 10 km; 1. O-line of AT; 2. isoline of AT(+); 3. isoline of AT(-); 4. isoline of gravity; 5. positive anomaly of gravity; 6. negative anomaly of gravity; 7. main fault; 8. deduced fault.

canyon into the center of the model. Take the fault to be elastoplastic, and measuring 0.5-1 km in width, other parts being considered as elastomer homogeneous in all directions. In order to keep t from moving, the bottoms of the model are bound. In the model, only lateral structural force is taken into consideration, and the impact of gravity is neglected. The results of calculations (Table 1, 2, Fig. 4) fit very well the earthquake sequence registered on the spot. The calculations also indicate:

The seismic events caused on the given geological model, under the impact of the stress field with the maximum stress axis of N60°W (lateral), the minimum stress axis (vertical), fit best with the actual earthquake sequence.

The stress field that resulted from these calculations is similar to that obtained by other means. The dislocation pattern of various faults obtained by calculation corresponds with the result of geological investigation on the spot. The dynamic parameters of some earthquake focuses obtained from calculation are roughly in accord with the focal mechanism.

Mathematical simulation demonstrates further that the fault of NNW strike in the vicinity of the main shock was the least stable before impoundment of the reservoir. The seismogenic structures of the main shock and the two after-shocks of a magnitude over 5 are of the same fault. After the main shock, the stress within the rock changed, both series of faults, NNW and ENE, had become unstable. The results of mathematical simulation correspond completely with the seismic activity in this area.

3 SHALLOW EARTHQUAKES AND "DEEP" EARTHQUAKES

The additional stress caused by the deformation of the lake basement is comparable to the stress status of a cross beam (Fig. 5). The stresses of the shallow layer and the deep layer of the lake basement are diametrically opposed to each other. There is a neutralized belt between them. This phenomenon coincides entirely with the focus distribution of this area. The induced earthquakes in the sphere of focus distribution can be divided into shallow and "deep" ones.
Fig. 4:


The deformation of the lake basement adds extension stress in the shallow layer and pressure stress at the deep layer on the margin of the reservoir. The additional stress, in certain cases, counter-balances partly the normal stress on the fault plane and reduces the friction of the fault plane. The additional pressure stress in the depth of the lake's margin increases the friction of the fault plane which has already risen steeply. This is unfavourable to the outbreak of earthquakes. But when the reservoir's release some of its water, the elastic basement of the reservoir rises towards its original position, thus adding a pressure stress on the shallow layer of the water area margin and an extension stress on the deep layer, leading to the outbreak of earthquake activity in the deep layer.

Fig. 6 shows that the frequency of shallow earthquakes is inter-related positively with the capacity of the reservoir, while that of "deep" earthquakes is interrelated negatively with it. The earthquakes of about 6 km in depth have no interrelation with the reservoir capacity.

The additional extension pressure is greatest, in the shallow layer of the water area margin, therefore earthquakes induced by the reservoir are concentrated there. The earthquakes in Hsinfengchihang Reservoir are concentrated in the canyon of the water area margin. Documentation shows that shallow earthquakes are distributed in the NNW direction, the same as the NNW fault, the deep earthquakes are
Table 1. The comparison between calculation and observed seismic effects

<table>
<thead>
<tr>
<th>Date</th>
<th>Magnitude registered on the spot</th>
<th>Magnitude and relaxed energy of earthquakes obtained by calculation</th>
</tr>
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<tbody>
<tr>
<td>Dec. 10, 1961</td>
<td>Ms4.1</td>
<td>Ms4.1-4.2*(1.10^{18} \text{ erg.})</td>
</tr>
<tr>
<td>March 19, 1962</td>
<td>Ms6.1</td>
<td>Ms5.8-5.9*(3.76*10^{20} \text{ erg.})</td>
</tr>
<tr>
<td>April 5, 1962</td>
<td>Ms4.9</td>
<td></td>
</tr>
<tr>
<td>July 19, 1962</td>
<td>Ms4.5</td>
<td></td>
</tr>
<tr>
<td>July 29, 1962</td>
<td>Ms5.1</td>
<td>Ms5.1-5.2*(3.54*10^{19} \text{ erg.})</td>
</tr>
<tr>
<td>Aug. 30, 1962</td>
<td>Ms4.8</td>
<td>Ms4.7*(10^{18} \text{ erg.})</td>
</tr>
<tr>
<td>Nov. 8, 1962</td>
<td>Ms4.8</td>
<td>Ms4.6*(5*10^{18} \text{ erg.})</td>
</tr>
<tr>
<td>Aug. 2, 1964</td>
<td>Ms4.6</td>
<td></td>
</tr>
<tr>
<td>Sep. 23, 1964</td>
<td>Ms5.3</td>
<td>Ms4.9-5.0*(1.73*10^{19} \text{ erg.})</td>
</tr>
<tr>
<td>Dec. 18, 1972</td>
<td>Ms4.5</td>
<td></td>
</tr>
<tr>
<td>July 25, 1975</td>
<td>Ms4.3</td>
<td>Ms4.3-4.4*(2*10^{18} \text{ erg.})</td>
</tr>
<tr>
<td>May 12, 1977</td>
<td>Ms4.7</td>
<td>Ms4.5-4.6*(4*10^{18} \text{ erg.})</td>
</tr>
</tbody>
</table>

Fig. 5. Sketch of the basement deformation caused by additional stress:
a. shows the horizontal displacement of basement in a section; b. shows the distribution of additional stress in a section; c. shows the distribution of additional stress in a block diagram.

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<table>
<thead>
<tr>
<th>Order</th>
<th>Date</th>
<th>Method</th>
<th>Magnitude</th>
<th>Dislocation amplitude</th>
<th>Strike of fracture</th>
<th>Dislocation mode</th>
<th>Max. stress axis in focal region</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>Dec. 10, 1961</td>
<td>Seismic</td>
<td>4.1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>N65°W</td>
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<td></td>
<td></td>
<td>Math. Sim</td>
<td>4.1-4.2</td>
<td>0.14 cm</td>
<td>N15°W</td>
<td>counterclock</td>
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<td>2</td>
<td>March 19, 1962</td>
<td>Seismic</td>
<td>6.1</td>
<td>14.5 cm</td>
<td>N28°W</td>
<td>counterclock</td>
<td>73°W</td>
</tr>
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<td></td>
<td></td>
<td>Math. Sim</td>
<td>5.8-5.9</td>
<td>28.26 cm</td>
<td>N10°-15°W</td>
<td>counterclock</td>
<td>N62°-70°W</td>
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<td>3</td>
<td>July 29, 1962</td>
<td>Seismic</td>
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<td>-</td>
<td>-</td>
<td>counterclock</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>Math. Sim</td>
<td>5.1-5.2</td>
<td>3.89 cm</td>
<td>N15°W</td>
<td>counterclock</td>
<td>71°-72°W</td>
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<td>Seismic</td>
<td>4.8</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Math. Sim</td>
<td>4.7</td>
<td>1.74</td>
<td>N30°W</td>
<td>counterclock</td>
<td>64°W</td>
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<tr>
<td>5</td>
<td>Nov. 8, 1962</td>
<td>Seismic</td>
<td>4.8</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Math. Sim</td>
<td>4.6</td>
<td>1.46 cm</td>
<td>N10°W</td>
<td>counterclock</td>
<td>65°W</td>
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<td>6</td>
<td>Sep. 23, 1964</td>
<td>Seismic</td>
<td>5.3</td>
<td>8.2 cm</td>
<td>N15°W</td>
<td>counterclock</td>
<td>25°E</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Math. Sim</td>
<td>4.9-5.0</td>
<td>1.64 cm</td>
<td>N30°W</td>
<td>counterclock</td>
<td>N69°-70°W</td>
</tr>
<tr>
<td>7</td>
<td>July 25, 1975</td>
<td>Seismic</td>
<td>4.3</td>
<td>-</td>
<td>N80°W</td>
<td>clockwise</td>
<td>70°E</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Math. Sim</td>
<td>4.3-4.4</td>
<td>0.76 cm</td>
<td>N67°E</td>
<td>clockwise</td>
<td>51°W</td>
</tr>
<tr>
<td>8</td>
<td>May 12, 1977</td>
<td>Seismic</td>
<td>4.7</td>
<td>-</td>
<td>N85°E</td>
<td>clockwise</td>
<td>50°W</td>
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<tr>
<td></td>
<td></td>
<td>Math. Sim</td>
<td>4.5-4.6</td>
<td>0.91 cm</td>
<td>N67°E</td>
<td>clockwise</td>
<td>52°W</td>
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</tbody>
</table>
Fig. 6. The correlation between the ratio of earthquakes in certain depths and the capacity of reservoir.

(Every time-section contains 127 days, from July 8, 1961 to April 19, 1964).

distributed in the ENE direction, corresponding with the ENE fault. Earthquakes of different depths have different seismogenic structures.

The water build-up in the lake triggers activity, releasing some of the stress already accumulated. Many factors probably contribute to this triggering. The seismic research work at the Hsinfengchiang Reservoir shows that the deformation of the lake basement caused by increased reservoir load and the resulting additional stress constitute a triggering factor for earthquakes activity which should not be neglected.

REFERENCES


INDUCED STATE OF SEISMICITY IN THE AREA OF A LARGE CHARVAK WATER RESERVOIR IN CHATKALO-KURAMINSK REGION

SISMICITE INDUITE DANS LA REGION D'UNE GRANDE RETENUE DANS LA ZONE DE TCHATCAL-KOURAMINE

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ABSTRACT

1. Variation in seismicity of region under the influences of Charvak water reservoir which is situated in a more seismically active zone of the pre-Tashkent district filling up of which commenced in 1971 and has been studied throughly in accordance with data obtained from a network of highly sensitive seismological stations which are functioning in this area since 1960.

2. Processes of filling up and exploitation of this water reservoir caused migration of the centres of earthquakes all over the given area.

3. Periods of filling up and exploiting of water reservoir are characterized by various natures of relieving of stresses which have accumulated in the earth-crust of the area in question. Initial period of filling up has resulted no alterations in seismic activity, while exploitation of water reservoir resulted in significant variations in seismicity due to increase in the amount of light shallow-focused earthquakes.

4. Cyclicality in emission of seismic energy becomes far more pronounced and the maximum of its energy fall on the beginning of every current year and corresponds to a period after its termination and its minimum for the specific period of the year upon level of water reservoir. The second maximum corresponds with the filling up period and reaching of the maximum water level in reservoir.

5. Relationship of parameters between the seismic behaviour and that of exploitation of water reservoir is of a non-linear nature and
this is proved by influences which a number of active factors play upon the state of seismicity (for instance gravitation, conditions of irrigation and so forth). A certain degree of tolerance in gradient value was produced in velocity of a change in the level which ensures decline in the effect of influence which water reservoir has upon the increase in seismicity of this particular area.

6. A detailed analysis was carried out into the nature of preparing and relieving of different stresses in course of natural tectonic (Tawasai M=5.0 XII, 1977) and induced (Dinapsk = 4.0 II, 1977) earthquakes in this specific region.

Construction of water reservoir has brought about a marked change in environmental properties and led to initial processes which culminated in a severe earthquake and this was preconditioned by a difference between the seismic port-ends of natural tectonic and induced earthquakes.

7. Taking into due consideration the shallow depth of different centres of induced earthquakes one is bound to expect in course of severe influences a rise in microseismic effect on account of high-frequency composite tremors and, as such, in course of planning and making estimates in regard to different hydroelectric projects for their seismic resistivity, along with taking into proper account of tectonic earthquakes of natural origin, one must invariable take into proper reckoning the high-frequency content of earthquakes of an induced nature.

ABSTRAIT

1. La variation de la sismicité de la région sous l'influence de la retenue de Tcharyk, située dans une zone la plus sismique près de Tachkent dont la mise en eau a été commencée en 1971, a été étudiée en détail selon les données d'un réseau de stations sismique fonctionnant dans la région à partir de 1960.

2. Le processus de la mise de la retenue en eau et son utilisation a provoqué une migration des foyers sismiques en terrain.

3. Les périodes de la mise de la retenue en eau et son utilisation se traduit par différentes caractéristiques de suppression des contraintes accumulées dans l'écorce terrestre de la région. La période initiale de la mise de la retenue en eau n'a pas provoqué aucune variation de l'activité sismique de la région. L'utilisation de la retenue a conduit aux variations importantes de la sismicité à cause d'une augmentation de nombre de petits et faibles tremblements de terre focaux.

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4. Une cadence en dégagement de l'énergie sismique se manifeste. Le premier maximum de l'énergie coïncide avec le début de l'année courante, et correspond à la période après l'utilisation au minimum, par l'année donnée, au niveau de la retenue. Le deuxième maximum coïncide avec la période de la mise de la retenue en eau et au maximum de son niveau.

5. La dépendance des paramètres du régime sismique en fonction du régime d'utilisation de la retenue porte un caractère non linéaire. C'est un témoignage de l'influence de divers facteurs exerçant leur influence simultanément sur la sismicité (gravitation, irrigation, etc.). On a défini la valeur limite du gradient de la vitesse de variation du niveau assurant la réduction de l'effet d'influence de l'eau retenue sur l'accroissement de la sismicité dans la région.

6. On a étudié en détail le caractère de la préparation et de la décharge de la tension aux tremblements naturels tectoniques (de Tavakasay M=5,0 XII. 1977) et induits (de Dinapse M=4,0 II. 1977) dans la région.

La création de la retenue a conduit aux variations des propriétés du milieu et du processus de la préparation d'un fort tremblement de terre. Ceci conditionne la différence des signes témoignant des tremblements de terre tant naturel qu'induit.

7. Compte tenu faible profondeur des foysers des tremblements de terre induits, il convient d'attendre, lors de forts influences, l'accroissement de l'effet macroseisique à cause des composants de hautes fréquences. C'est la raison pour laquelle lors d'établissement d'un projet et des calculs des ouvrages hydrauliques au point de vue de la résistance aux seismes, il convient de tenir compte non seulement aux tremblements de terre tectoniques naturelles, mais aussi à ceux des composants de hautes fréquences des tremblements de terre induits.

During construction of high dams in different seismically active areas as well as building of large-scale artificial water reservoirs, along with a number of engineering and geological problems one is usually confronted yet by another problem, viz. the influence such water reservoirs have upon the seismicity of the specific region and, as a matter of fact, this problem is so complicated, that it requires a good deal of a systematic and comprehensive analysis because its successful solution proves to be of vital interest from two perspective points of view.

Of these, the first point is closely associated with matters which relate to proper evaluation of seismic danger that is always present in areas where hydro-technical projects are being planned and far deeper comprehension of physical properties of probable...
centres of an earthquake, while the second aspect of this problem relates to successful solution of seismic resistivity of dams in question as results of these investigations give us a chance to properly evaluate the reaction such projects have upon the actual seismic influences and to effectively draw a comparison with theoretically prepared calculations in respect to existing estimates in regard to diagrams of seismic resistivity.

In world-wide practice we are quite aware of scores of cases when in certain areas changes were observed after filling up of water reservoirs or, for that very matter, effective state of seismic tremors took place and in number of instances tremors proved to be of a most violent intensity (for example in the regions of such dams, like Kariba in Central Africa where amplitude of earthquake reached the intensity of $M=6.5$, $J_o=8$ points, or Cremasto in Greece where its magnitude was estimated at $M=6.3$, $J_o=8$ points, or at Koyna in India where it reached $M=6.5$, $J_o$ between 8 and 9 points, or at Hsinfengkiang (China) where intensity of earth tremors registered $M=6.1$, $J_o=8$ points, as well as a number of other places). It's quite of interest to note, that during these tremors serious damages occurred in quite a number of inhabited centres which brought about devastation and took away many human lives and, if one takes into due consideration that such areas, like Koyna and Kariba, were hitherto regarded as non-seismic, then it tends to be quite evident as to how important the problem of maintaining a strict control over the seismicity of different areas in which hydro-technical projects are being erected in reality is.

At present, in different seismically active districts of the USSR (such like Central Asia, Caucasus, etc.) where high dams are being erected specialists are conducting seismological investigations with a view of effecting proper-control of seismic behaviour of the region in question as well as exposing their actual causes and aftereffect to study the mechanical–properties of earthquakes which are prompted in the regions of water reservoirs. Ultimate results of investigations of this nature will help us to forecast time of occurrence of severe earthquakes which presents grave danger to hydro-technical installations and side walls of water reservoirs.

In certain parts of Central Asia, where seismic magnitude reaches an intensity of 8 or 9 points, we are at present executing, or have already completed, construction on a number of large dams, like at Novsk which has a height of 300 M, or the one at Tagtogul with a height of 120 M, the one at Charvak being 170 M in height, Andizhan–120 M and the latter pair of dams are situated upon the territory of Usbekistan.

Hydro-electrical project at
Charvak, which is some 65 km to north-east of Tashkent, is in one of the most seismically affected parts of the district, is constructed of rock fillings and is embedded upon a rocky foundation. In making calculations in regard to its seismic resistivity it was decided that the maximum possible intensity of an earthquake in these parts will not exceed 8 points at the most.

Filling up works on water reservoir at Charvak began in 1971 and its rated capacity is estimated at 2 km$^3$ and its total area will cover 40 km$^2$ with an overall distance of 12 km and a width of some 4 km. In 1971 its mean water level reached a height of 77 m, in 1974–75 it rose to 110 m, while during the 1978–1980 period it rose as high as 145 m. Annually this water reservoir is filled up during the months of March and April and its volume is increased each year by three or four times its initial storage capacity. It remains filled to its full capacity for a period of 1 - 1.5 months, i.e. during the May - June period while in course of the following 2.5 - 3 months it is drained steadily until it reaches its average level of 75 to 80 m which is maintained from August till March of the next year. The only exception to this regularity was made during the period 1978–1980 years when its steadily maintained level of 145 m was kept up in course of 9 months (from May) effluence was effected in course of some 2.5 months only. After this procedure the reservoir was again filled up to a height of 145 m.

District, in which this water reservoir is located, is characterized by its complexity of geological and tectonic structure since it is situated at a point of convergence of two sub-zones with a different history of geological development, viz. the Chatkalo and Kurasinsk zones (fig. 1).

Fig. 1: Diagram of geotectonic region-wise layout of Chatkalo-Kurasinsk area (compiled by V.A. Arapov, V.V. Kozyrev, V.P. Kozhemyakov, et al.).

Energy class: K=15 E, J:
1 - K=11, M=4.0; 2 - K=12, M=4.5;
3 - K=13, M=5.0; 4 - K=14, M=5.5; 5 - K=15, M=6.0.
6 - Profile of the area of Charvak water reservoir.

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7 - Seismic stations.
8 - Axial lines of deep fissures.

The region is intersected by tow regional faults—the Kenkol and the Kounbel and the certain tectonic structures of a less significant order are also present, like the Ishak-Kupriuk Charvak and the Brichmullah ones which encompass the boundaries of the reservoir bowl. Western border of the reservoir is in alignment with the Charvak upthrust where it becomes contiguous with paleozoic limestones and tertiary deposits (amplitude of displacement is in the range of 800 m). Northern boundary runs along the Karzhatanau upthrust which is regarded one of the largest broken disruptions of this area (amplitude of displacement is equal to a distance of 2 km). Its western boundary converges with the Brichmullah upthrust (amplitude of displacement is 2 km) and, ultimately, its southern boundary is encompassed by the Chimgan upthrust with displacement of 1 km. The bowl of Charvak reservoir is situated within the limits of Brichmullah depression and, along with its sides, comprises of Neocene cretaceous and Quaternary period. Thickness of the tertiary deposits within the depression area is equal to some 1300 – 1500 m.

Aquiferous horizon of free-flowing waters exists at a depth of 20 – 150 m in the limestones of Paleozoic period and ground waters in Quaternary deposits are timed to shingly rock species and deluvial sandy loams and are bedded down upon the threshold with practically water-resting species of tertiary period. Effluence of waters out of protective cover of tertiary deposits was not found anywhere.

Out of the 42 earthquakes of high intensity (M≥5) in Tashkent area 14 belong to Charvak region a proportion of some 32%. Earthquakes have occurred within the bounds of Charvak region during which shocks on the surface of the earth reached severity of 8 points and more. The epicentre of Brichmullah earthquake occurred in 1959 (M=5.8 (K=14), J6 = 7–8 points) was located within the north-easterly part of Charvak reservoir.

Seismologic investigations into studying the effects of Charvak reservoir which is undergoing its construction upon local seismicity were started in 1970 (V.I. Ulomov, L.M. Plotnikova and Yu.M. Bezrodny). Specific features of these seismologic investigations in conformity with the evaluation of effects which artificial reservoirs have upon the seismic activity of each region is in thorough analysis of seismic regime in course of prolonged time interval (I).

Different periods were carefully studied: the one preceding the construction of the dam, i.e. 1898–1970; period before filling up of the reservoir (1961–1971); and the period of filling up and exploitation of it (1973–1980).

The main objective of those
investigations were focused on:
1) Close study of the nature of parametric variations in seismic regime and dynamics of earth's crust of this specific region due to filling up of its reservoir; 2) Establishing of correlational relations between the actual conditions of exploiting the reservoir and appearance of seismic irregularities; and 3) Genesis and prediction of strong earthquakes within the area of such reservoirs.

In accordance with the complex programme of investigations which have been initiated in this region since 1969, we are conducting high-precision geodesic investigations here with a view of properly analysing deformations which may occur in earth's crust and levelling operations of class 1 category were carried out around the reservoir on 5 different occasions, viz. in 1969, 1970, 1972, 1974-75 and in 1978, while in separate areas repeated levelling operations were carried out in 1976, 1977, 1979, and 1980 and from data thus obtained we find out that filling up of this reservoir up to H = 90 m level has resulted during the 1969-1974 period to its sinking by 2 cm from earth's surface in the north-eastern and elevation in the western parts of reservoir (N.A. Koreshkov and A.P. Raisman). Continuous sinking in the eastern as well as north-eastern parts of this reservoir were observed by means of surveys made in 1980 and, in all probability, is closely associated with rise of water level up to a height of 145 m.

It was established in course of the 1969-1974 period by means of electronic distance-measuring equipment that the ranges which surround this reservoir on its eastern and western banks, are converging at a pace of 0.5 ± 0.2 Oh per annum. It's worth-while to mention here that, through series of geodesic investigations which were undertaken in the Inguri area of HEPS (Daghestan), it was likewise established that the level of water in this reservoir has fallen in the regions of dam alignment area. At this, the most pronounced camber was marked at the reservoir's bottom (upto 11 mm) and the least at the canyon edges (I.P. Kuzin of the S.Y. Zhuk HYDROPROJECT). In this case deformation of the reservoir bed occurred with a certain amount of lag in its periodicity as related to its maximum water table. As regards the area around the Toktogul reservoir, deformation of its bowl occurred coherently with the gradual filling of reservoir (V. Pavlov, IPZ AN of the USSR). Thus, a network of seismic stations, which are located all over the region, as well as the employment of local hodographs permits us to determine proper coordinates of different hypocentres of ensuing earthquakes with precision up to ± 2.0 km.

Epicentres of severe earthquakes which occur in the Charvak area are associated mainly with tectonic faults, and, in particular,
to zones of their intersecting (for example, Charvak reservoir is located in the vicinity of one of such faults, see fig. 1).

Comparison diagrams of different epicentres of representative earthquakes which occurred during the 1960–1970 and 1973–1980 periods (fig. 2) indicate that filling up and exploitation of reservoirs served as the primary cause of occurrence of slight earthquakes intensity of which was equal to M=1.2 at a depth of 5 km within the limits of the Alizorsk depression (at a distance of 3–7 km from dam alignment site) where prior to this no earthquakes were registered by seismic stations (a local network of such stations has been operating in this area ever since 1960). A most pronounced activity since 1976 in form of slight (2,5 ≤ M ≤ 1) and strong (3,0 ≤ M ≤ 4,0) shocks, associated with influences displayed by the reservoir, is detected along the north-easterly tongue (along the Pakem river bed) which gives us a possibility to presume presence here of a fault which borders on the Sidjak anticline and, as a proof of this deduction, we can bring data of geodesic surveys which have produced the opposition of signs of banks displacement within the area of this section of reservoir. Slight shocks were detected in its bowl as well.

Migration of the hypocentres of earthquakes was evidenced all along the area in course of filling up of the reservoir and, if in course of the initial period of

![Fig. 2: Map of epicentres of representative earthquakes of Charvak region for 1973–1980 period.](image)

Energy class: 1– K=6–7, M= 1,0–2,0; 2– K=8, M=2,0–2,5; 3– K=9, M=3,0; 4– K=10, M= 3,5; 5– K=11, M=4,0; 6– K= 12, M=4,5.

Depth of hypocentres: 7– H= 0–5 km; 8– H>5 km; 9– H=10 km; 10– H=15 km; 11– H=20 km.

12– Seismic stations.
13– Outline of the reservoir.
14– Alignment of the dam.
estimated at J=6 points, M=4.0 and its foreshock of M=3.6 occurred within the precincts of the north-ea-
tern part of the reservoir in the region of Brahmlala-Pakeam fault. Upto 1971 seismic activity dissi-
mated itself quite evenly along the depths of a layer with thickness of 5 -15 km. However, filling up of the reservoir resulted in decrease of earthquake focus depth and to activisation of the earth’s crust of 2 – 5 km thickness and, as was noted by a number of investiga-
tions, a slight depth of hypocentres of induced earthquakes appears to be quite typical factor for them.

In its primary approach, state of seismicity in this region may be characterized by a number of shocks and the total quantity of seismic energy emitted in course of this process, while on the other hand, these parameters may be used just as volume factors of deforma-
tions surveyed.

Specific feature of seismicity in course of the 1973-75 period (at H=100m), i.e., the initial period of exploitation of the reservoir was analysed on respect of data obtained for the 1960-70 period before filling up operations began on the reservoir. During this time annual samplings, which included earthquakes of M=1.0-5.0 (fig.3) were carefully analysed and it was established that seismic regime, as expressed in form of a number of tremors of M= 2.2-5.0 (K=8-13) throughout the entire duration of analysis period was of sufficient-
ly immobile nature. Very insignificant rise of activity was observed during the 1965 and 1967 years, while at the beginning of filling up of the reservoir (1972) produced no progress in seismic processes on the diagram. Period of exploitation of reservoir is characterized by somewhat stationary level of seismicity, while a certain amount of activity was observed in 1975 as in 1977, but the general state of seismicity, as expressed by the amount of shocks with M=2.0-5.0 was maint-
tained throughout the 1963-1980.

Entirely a different picture presented itself during analysis of samplings of light earthquakes with M=1-2 (K=6-8) and M=1.7-3.0 (K = 7-9). In this case rising tendency shows itself quite identical since 1973 by amount of small shocks which, evidently, were conditioned by filling up period of the reservoir. Having this factor in view, the initial stages of filling up of reser-
voir neither had any material effect on altering progress of seismicity in this particular area, while decrease in amount of shocks in 1977 is closely associated with relief of elastic stresses in the earth’s crust due to appearance in this area of severe earthquakes (on March 15, 1977 M=4.0 and on December 6, 1977 M=5.0).

A more detailed state of seismicity during exploitation period of the reservoir was studied through series of observations made after much shorter time intervals, i.e. in accordance with a monthly du-
Fig. 3. Distribution of number of earthquakes according to time lapse depending upon energy class: $K = \log E$, $j$: Periods: 1960-1970—prior to filling up; 1971-1972—filling up period; 1973-1980—reservoir exploitational period.

seismic energy released.

Likewise periodicity of energy release has been observed, i.e. one maximum of it is allocated for the beginning and the second—for the middle of the current year. First maximum corresponds to its minimal value for water level during the given period which holds steady in course of 7-9 months (a period, calculated subsequent to commissioning reservoir into exploitation), while the second corresponds to period of filling up and reaching of maximum water level (it lasts for 1-2 months). Seismic activation, which corresponds to the first maximum, makes itself evident after a period of 4-5 months following decrease in storage capacity of reservoir. At this stage large portions of energy are closely associated with the occurrence of strong earthquakes which usually coincide with periods which follow decrease in holding capacity of respective reservoir. During this time there appeared no alterations in amount of weak tremors throughout all the series of observations during 1973-1980 period. In 1976 this reservoir was filled up to the height of 125 m and its holding capacity was steadily reduced to a level of 85 m in course of one-and-a-half months at velocity gradient of effluence of 1.0 m diurnally which resulted to appearance in the north-easterly part of reservoir after a lapse of 6 months (in the second and third quarters of 1977) of two strong
Fig. 4. Correlational curves of variability in time lapse of number of earthquakes and total seismic energy released depending upon rates of exploitation of Charvak reservoir.

earthsquakes of $M=3.6$ and $M=4.0$ ($J_a=6$ points) (2). In course of the subsequent period in 1977 no second maximum activation was observed which was associated with filling of the reservoir up to the $H=110$m level and since 1977, after the aforementioned severe quakes, one could notice disruption of seismic activity which was induced by the rate of reduction in water-level of the reservoir. In May 1978 a designed level of $145$m was reached and was maintained practically up to August 1980 (velocity gradient of effluence was about 0.2m diurnally). This specific period of time was noted by decline in seismic activity in this area and insignificant rise in portions of energy, which was noticed in April 1979 and in August 1980 was induced by fall in holding capacity of the reservoir (velocity gradient of effluence 0.6m diurnally). Noted disruption in regularity of seismic movements is probably associated with relief of a substantial portion of stresses that occurred at the beginning of 1977 in course of alteration in velocity gradient of effluence which was equal to 1m diurnally.

Mean protracted value of inclination angle of recurrence diagram of earthquakes for Charvak region is estimated at $\gamma=0.43$, while for the 1960-1971 period it was $\gamma=0.41$. During water level reduction period of this reservoir (1973-1980) its value increased to $\gamma=0.56$ which, in all probability, may well be associated with effects it had upon the surroundings (3). Analysis of $\gamma$ value fluctuations for the annual periods of time indicate

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that the beginning of reduction in capacity of the reservoir has brought about disruption in correlation of the amount of earthquakes of different energetic classes. Thus preparation of an earthquake of M=4.0 (in February 1977), located in reservoir area, was characterized by abnormal increase of angle up to $\gamma = 0.55$. It is worth-while mentioning that, in existing conditions of the area, prior to construction of the reservoir here, preparation of a natural (not induced) tectonic earthquake was characterized by decline of the angle to $\gamma = 0.28-0.35$ as compared to mean protracted value of $\gamma = 0.43$ (3) and it is not excluded, that preparatory conditions of natural tectonic quakes and induced ones are quite different and associated with alteration in properties of invermmental media due to various influences of the reservoir (such as gravitation or filtration). This may be proved by changing of ratio between P- and S-waves velocities $V_p/V_s$ which is interconnected with variations in stressed condition of rocks during the preparation period of earthquakes; when filling up the reservoir it is also connected with additional loads and filtration of liquids into layers of earth's core. For structures which directly adjoin the reservoir (within the radius of 30km), in 1974-1975 we observed a substantial rise in $V_p/V_s$ as compared to 1972 and the mean value of it for Charvak $V_p/V_s$ values varied from 1.64-1.67 in 1973 to 1.80 in 1975, while in case of remote structures (30km and more) no increase in $V_p/V_s$ was noted during this period (2).

At the time preceding filling up (1969-1970), the focal mechanism of Charvak earthquakes was governed by general regularities typical for the entire area of the orogenic part of Chatkal-Kuraminsk region (4). Principal movement in earthquake focuses is upthrust. The direction of the plane of rupture coincides with the one of basic geological structures, while contraction axis is spearheaded towards the cross-sectional part of structural extension. The axis of dilatation retains practically vertical posture. An exception to this was made by only two earthquakes. One of these ($M=3.3$) occurred on November 27, 1968 and relates to a period of preparatory operations in the foundations of the dam (more than 40 metres of quaternary and rocky species were removed then) and boring-cum-blasting operations at the quarries for procurement of building materials. This earthquake occurred 5km far from the site and its focal mechanism corresponded to the rise of the Western sector of this territory which was later found out by means of a geodesic survey. The second earthquake ($M=3.6$) with its anomalous focal mechanism occurred in February 1977 following an intensive effluence of water on its reaching the maximum (for this period of time) level of 145m in 1976. A month later in March 1977
another, much more intensive quake \((M=4.0, J=6)\) took place and its focal mechanism appeared to be quite typical for this area. Anomalous nature of afore-said earthquakes focal mechanisms was expressed in the alteration of the direction of its cross-sectional rupturing plane while that of contraction axis along geological structures with retaining its upthrust displacement in the focus and its dilatation axis unchanged. Re-distribution of the components of the stress field in the region is probably associated with relieving of stresses in earth's core as a result of scavanging operations and abrupt release of extra vertical load. The rise of the Western sector of Charvak reservoir in 1969–1974 serves as indirect assertion of such deduction. A more correct solution to the point of existence of interrelation between the exploitational regime of the reservoir and seismic activity is based upon definition of quantitative characteristics with application of statistical methods of interpretation. 

Regime of exploitation of the reservoir we describe by water level \(H\), velocity of variation in level \(\beta = \frac{\partial H}{\partial t}\), and velocity gradient with due consideration for water column pressure \(\frac{\partial}{\partial z}\) and \(\text{grad.BH}\). Seismicity is described by number of earthquakes \(N\), summary seismic energy \(\Sigma E\) and deformations relieved \(\Sigma V E\) in a time lapse.

On strength of data published it appeared that for different regions there is no identical temporary dependency between the rise in water level and regime of reservoir exploitation and seismic activation \((5,6,7)\). Coincidence of phasic recurrence of such phenomena has been noted in some regions, while in others a definite bias was observed \((3-6 \text{ months})\). This may depend both on the geological and tectonic conditions, degree of stress state of earth's crust, lithology of the bowl, and the dimensions and regime of the reservoir.

Keeping in view seasonal level variations of Charvak reservoir the material processing was made through variable time interval \((1, 4, 6, 12 \text{ months})\). As a result of this different character of dependency was established in seismic parameters under review for periods associated with filling up and exploitational periods of the reservoir. Correlational coefficient thus obtained indicate to absence of any type of a linear dependency of seismic parameters \((N, \Sigma E, \Sigma V E)\) upon water level at \(H=90-100\text{m}\). For the filling up stage \((1973-1974, H_{\text{max}}=90\text{m})\) direct dependency of seismic parameters existed only from parameter \(\frac{\partial}{\partial z}\).

Values of correlational relations for various cycles of reservoir regime are evaluated and equal respectively to: for filling up – \(\eta = 0.60-0.86\), for effluence – \(\eta = 0.55-0.91\), which stands to prove the non-linear dependency of the parameters \(\Sigma E\) and \(\Sigma V E\) from \(H\), \(\beta\) and \(\frac{\partial}{\partial z}\).

The obtained results show,
that the reservoir influence on
the properties of the rock species
within the radius of 30–35km (area
of 5000km$^2$).

CONCLUSION. Studying of seis-
mic activity in the area of the la-
argest reservoir in Uzbekistan loca-
ted in Chatkalo-Kuramin region has
shown that:

1. The character of seismic ac-
tivity during periods that precede
construction (1960–1970), fill up
(1970–1972) and exploitation (1973–
1980) of the reservoir is quite
different. Within the third period
space distribution of earthquake
focuses is regulated by structural
factors and exactly correspond to
the faults surrounding the earth-
crust block within which the rese-
voir is located. At that period the
largest number of quakes (including
strong ones, $M=4.0$, $J=6$) occurred
in the north–easterly part of the re-
servoir, 12km far from the dam
alignment, in places waterrepellent ter-
tiary rocks did not exist and fiss-
urized Paleozoic deposits predomi-
nated.

2. Rise in water level in Char-
vak reservoir up to $H=145$m doesn’t
promote seismic activisation by way
of increasing the number of shocks.
This proves indirectly the absence
of poric pressure on this area. In
conditions of Charvak region se-
ismic activisation is influenced by
value of velocity gradient of alter-
ing water level in the reservoir
(1$m$ diurnally) and shows itself
with temporary delays which is de-
termined by degree of constituency
of rock species which form the
bowl and the banks of the reservoir.

Interrelation between the se-
ismic parameters exploitation re-
gime of the reservoir is of non-
linear character. This substantiates
the appearance in due time of a
more complex set of interrelations
caused by influence of several si-
multaneous active factors (such as
gravitation, filtration, etc.). Cha-
acter of correlational relations
evidence that gravitational factor
predominate in environmental condi-
tions of Charvak reservoir.

3. Space distribution and va-
riation in time lapse of $V_p/V_s$
relations, anomalous focal mechanism
of strong earthquakes, the very na-
ture of releasing seismic energy,
of deformation and distribution of
the mean protracted regularity of
distribution of total number of
earthquakes in accordance with the-
ir energy testifies as a whole of
redistributive properties of com-
ponents of the stress field and
changes in properties of surroun-
dings environments as well as the
actual process of preparing strong
earthquakes within the area sub-
jected to influences of Charvak re-
ervoir; and such circumstances
serve as prerogatives to differen-
tiate seismic heralds of severe
reservoir type and natural tecto-
nic earthquakes which should be
accepted into close reckoning when
working out relevant forecasts
pertaining to forthcoming earth-
quakes.

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4. Investigations made upon the influences the reservoir has upon the seismic activity of the region, production of the relevant factors, effects of which provide for a more pronounced possibility to work out recommendations on the optimal rate of exploiting reservoir, thus reducing its effects and ensuring high seismic resistivity for a towering hydrotechnical project.

5. In taking into account the shallow depth of hypocenters of induced earthquakes one should always anticipate excitation of high frequency effects and the latter will result in rise of macroseismic effects at the very same intensiveness of its original source. As such, in course of drawing plans and calculating of a hydrotechnical project as to its seismic resistive capacities one should take into proper reckoning the influences of not only natural tectonic earthquakes may have upon such project, but various corrections that should be introduced owing to high-frequency actions that may be brought by induced earthquakes and possibility of appearance of gravitational waves as a result of primary and secondary residual deformations.

References


Seismicity Changes Associated with Koyna Reservoir—An Area of Suitable Tectonic State for Stimulated Seismicity

Des changements de sismicité associés au réservoir Koyna—Une région d’état tectonique convenable pour sismicité stimulée

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Abstract

Following the initial impoundment of Shivajisagar lake behind Koyna dam, India, in 1963, continued seismic activity has been observed in the area. This is contrary to the low seismicity level by and large prevailing in the shield area. The epicentres are located close to the reservoir and focal depths are shallow. Moderate to high seismic activity, at least upto 1971, strongly correlates with the lake level fluctuations and the period of maximum water level retention.

The earthquake energy is considered to be of tectonic origin with the artificial stress acting as a triggering stimulus. The unique volcano-tectonic setting of this part of the subcontinent provides a high and critical stress regime basically required for seismicity to be induced by increased pore water pressure.

Abstrait

Suivant la mise en fourrière initiale de l’étang de Shivajisagar derrière barrage Koyna, Inde, en 1963, une activité continue sismique a été observée dans la région. Il est à contraires au niveau sismique bas qui domine à tout prendre à la région d’écou; les épisentres sont situés à la proximité du réservoir et des profondeurs focales sont de nature superficielle. Modéré en activité sismique haute, au moins jusqu’à 1971, s’accorde fortement avec des fluctuations du niveau d’étang et la période de retention au niveau d’eau maximum.

L’énergie de tremblement de terre est considérée d’être de l’origine tectonique avec la force artificielle fonctionnant comme un stimulus détente. La composition unique volcan-tectonique de cette partie du subcontinent fournit un régime d’accentuation haute et critique, fondamentalement exigé pour sismicité ce qu’on doit persuader par la pression d’eau de pore amélioré.

1. Introduction

Moderately high seismic activity of the Deccan plateau, specially of the Western Ghats region, was first recognised on December 10, 1967, when an intensity IX (MM) and M 6.5 earthquake took place in the region around the Shivajisagar lake behind Koyna dam. This devastating earthquake, resulting in deaths, injury and extensive loss of property, was felt over 698 x 10⁶ km² (Mukherjee 1971) of which the main seismal area was 48 km² including 3 km² of intensity +VIII around the main dam in Koyana (17°23’N; 73°45’E). Available records of historical seismicity indicate that this region experienced moderate intensity earthquake shocks of local nature during the past 350 years. Such activities never culminated to a major event before 1963, the year of initial impoundment of the Shivajisagar lake (volume 2.8 x 10⁹ m³ and maximum depth 103 m). Because of lack of detailed information about the pre-reservoir seismicity of this region, it is not
possible to establish the exact nature of change of seismicity level associated with reservoir filling. However, post-reservoir seismicity (after 1963), specially the Koyana main shock accompanied by long foreshock and aftershock sequences, may be considered to be representing the change of seismicity level in this area.

2. PRE-RESERVOIR SEISMICITY OF KOYNA REGION

The southern peninsular region was by and large considered to be aseismic because of the mild and local nature of the majority of tremors in this region. There had been stronger shocks as those experienced near Bellary (1843), Coimbatore (1900) and Vizianagaram (1959) which were felt over considerably large area. The recorded epicentres of these shocks lie in the central part of the crystalline Precambrian shield.

Gutenberg and Richter (1954) based on their global seismicity studies, concluded that the ancient shield area are seismically stable except for their marginal areas and rift and fault zones present within. The Indian peninsular shield was also shown as merely aseismic in their global seismicity map.

Reports of local inhabitants and project personnel, for the period before 1962, indicate that the seismicity of the Koyana region is in tune with the generally conceived aseismic nature of the peninsular region. This is corroborated by the records of sensitive Geiloff seismometer of the Indian Meteorological Department, located at Poona about 120 km north of Koyana dam and operating since 1950, which has not registered any significant tremors in this region during the period before 1962.

3. POST-RESERVOIR SEISMICITY OF KOYNA REGION

Many small shocks began to occur in the immediate vicinity of the Koyana dam following the initial filling of the reservoir (Shivajisagar lake) in 1963. Isolated events of magnitude less than 3.7 continued till September 13, 1967, when an earthquake of magnitude 5.8 occurred. This seismicity level, hitherto unknown in this region, soon culminated in a large disastrous event (Koyana main shock) of magnitude 6.5 (M 7.0 CUPRS data) on December 10, 1967, with epicentre very close to the right abutment of the dam. Since then the earthquake magnitude and frequency are on the decline obeying the usual after-shock sequence laws interrupted by occasional spurts of activity. The concrete gravity dam, which developed minor cracks at places, was not designed for such a shock and was strengthened by buttresses in view of possible recurrence of similar events in future.

During the post-December 1967 period the Koyana region and its neighbourhood has experienced 46 moderate magnitude (4.0M) events besides a significantly large number of microseismic totalling about 30,000 shocks of magnitude above 1 (Guha et al 1974). In 1980 tremors of somewhat little more severity than usual occurred on February 7 (M 4.2), and specially in September when three moderate size events occurred within a short span of 13 days. First event (M 4.3) of this series occurred on 2nd September and was feebly felt between Bombay and Goa. The remaining events (M 4.7 & 4.9) occurred on 20th September and were very conspicuously felt over an area of 250 km in radius around the epicentre located in the headwater reaches of the Wanar valley, a little east to the south of Koyana reservoir. The occurrence of these shocks in quick succession, contrary to the general decay trend of seismic activity in this area, and migration of strain focus towards south in the Wanar valley, where a large dam is under construction, have generated a feeling of deep concern.

4. KOYANA NET

There can be no doubt that a distinct change in seismic regime of the Koyana region was experienced following the initial filling of the Shivajisagar lake in 1962-63. This led geoscientists (Krishnan 1967) to believe that, like classic examples of reservoir induced earthquakes of Lake Mead, Marathon, Kariba and Kremasta, the Koyana tremors are also related to reservoir loading. Therefore, for detailed investigation of changes in seismic regime associated with the filling of Koyana reservoir, a small, dense and independently working network of seismograph stations (Koyana Net) was installed under the control and supervision of the Central Water & Power Research Station (CWPERS), Poona. Installation commenced in early sixties in response to tremors following initial filling of Koyana reservoir. Now it is an organised network of 12 stations located between latitude 16°49' and 18°32' and longitude 73°29' and 74°39'.
5. RESERVOIR LEVEL AND EARTHQUAKE FREQUENCY

Earthquake shocks and water level data for the 5-year period 1963-67 was studied by Gupta et al (1976) who inferred that tremor frequency in Koyna region is related to the rate of increase of water level, maximum water level reached and retention period of the high water level. A statistical analysis of correlation between the tremors and the levels of Shivajisagar lake for the 10-year period 1964-73 is available from the studies of CWPRS group of seismologists (Guha et al 1974). Fig 1 shows the water level and fortnightly tremor frequency plotted on the same time scale for the period 1964-80 (CWPRS data presented during the 10th meeting of the Koyna Tremors Committee, October 1980). The time series distribution of dam site shocks does not appear to be random. On the other hand it exhibits a periodicity coinciding with the months of higher water level during July to November. Notable periods of enhanced seismicity, in a descending order, are August-December 1967, September 1968, October 1973, September 1977 and September 1980. It is to be noted that all these periods are following the rainy season when the highest water levels are attained although retention period of high water level varied from time to time.

The studies (Guha et al 1974) of earthquake residual peaks for most of aftershock sequences in this region particularly during 1968-71 show a return period of 24 to 30 fortnights (roughly a year) indicating that they are largely influenced by annual water level fluctuations. Correlation co-efficient between the water levels and the earthquake residual peaks is as high 0.80 with little time lag indicating clear influence of water load. The earthquake residual peaks for the post-1971 period however show less significant correlation with lake levels. Although the water level was maximum in 1973 and retained at 658 m for more than three months the seismic activity was less severe when compared with the events of August-December 1967. Chauhan and Gaur (1971) based on their studies of strain accumulation and release curve for shallow earthquakes of Koyna and adjoining areas for the period of 1950-67 concluded that all strain energy accumulated in this region had been used up or released by the Koyna main shock.

6. TECTONIC STATE AND ITS IMPORTANCE

It is now generally accepted that loading by reservoir of sizeable dimension, may, in a manner like fluid injection, underground mining and detonation of nuclear explosions, fluid extraction, etc., modify the existing tectonic state and enhance the likelihood of seismic activity. The change in the stress regime brought about by the reservoir filling are considered to be too small (Cather 1970, Gough & Gough 1970, Higurawa & Ohtake 1972, Raleigh & Healy 1973, Snow 1973) to induce seismicity in a lightly stressed region. The general consensus of opinion, emerged from the studies of more than 40 cases all over the world, indicate that
seismicity associated with filling of large reservoirs is a triggered effect on a critically stressed crustal volume. The inferences, of modifications of tectonic state due to impoundment, drawn either by fluid communication hypothesis (Healy et al 1968) or by crustal loading hypothesis (Gough & Gough 1970), are based on tacit assumption that the stress condition prior to impoundment are critical.

Thus it is evident that, very high ambient tectonic state, is an essential prerequisite for earthquake activity to be induced by reservoir loading. An important and very significant observation in this premise is that such phenomena are not known to occur irrespective of regional tectonic style of the area concerned. Large reservoirs like Shakra, Bass and Ramganga, in low angle Cenozoic thrust faulting environment in the Himalayas, had no observable seismic activity following initial impoundment. Observational evidences and simple logic, of reservoir induced earthquake mechanism based on Mohr-Coulomb criteria of shear failure, suggest that such phenomena is most probable in geologically recent normal or strike-slip faulting environment and least likely in thrust faulting environment.

Naturally existing tectonic state and style may, therefore, be considered as a potential factor to cause failure of the lithospheric crust and resultant earthquake activity when acted upon and modified by external loading like filling large reservoirs. Thus it can be logically concluded that seismic activity related to the reservoir impounding is equally related to the tectonic state. As for Koyne where induced seismicity was of largest magnitude (M 6.5), Lee and Raleigh (1969) concluded triggering of large pre-existing tectonic strain and Sykes (1970) opined of very high to extensive horizontal compressive stresses in the lithospheric plate in this region.

7. TECTONICS OF INDIAN SHIELD

The tectonics of Indian shield is to be viewed concurrently with the northward drifting of the Indian plate. The west coast represents a deep continental rupture when India-Madagascar-Antarctica block splitted from Africa with the initial opening up of the Indian Ocean during Mid-Jurassic period. The Ninety-east ridge, Laccадive-Chagos ridge and Owen fracture zone, which run across the mid-oceanic ridge systems of the Indian Ocean floor, are the imprints and indicate northward drifting of Indian plate at least in its last phase (Heezen & Thrup 1965). The stupendous volcanic activity of the Deccan plateau is related to the continental splitting and subsequent northward motion of the Indian plate across a thermal centre or hot-spot (Dietz et al 1970) in Indian ocean which is responsible for the continental type spatial distribution of the great volcanic province.

A striking geomorphic feature of the Indian shield is the east-west trending 1400 km long Narmada-Son rift. This has resulted from the tendency of the extratropical crust to upwarping (Ghosh 1976) or buckling and may be termed as 'Narmada-swallow-and-rift' (Bose 1973). The gravity map (Kailasan et al 1972) of Indian shield area suggests a linear gravity hump along the Narmada valley followed by gravity low further south. The buckling or upwarping tendency of the lithospheric crust may be attributed to the continuous accretion of Indian Ocean floor at a faster rate than the rate of plate consumption at the Himalayan subduction zone. A similar feature is anticipated in the Ceylon-Cocos zone of the Indian Ocean where the anomalous high seismic activity, compared to other ocean basins, is attributed to the tendency of lithospheric upwarp (Sykes 1970) and possible formation of nascent island arc to accommodate the increasing plate dimension.

Within the tholeiitic Deccan basalt province there is alkaline magmatism along some well defined belts in the Indian shield. This alkaline carbonatite magmatism is represented by plugs and cone like bodies emplaced along four arms intersecting near Gulf of Cambay. Thus the Cambay region represents a four-arm rift junction, defined by Rajasthan-Gujrat rift and West Coast rift in a north-south direction, and Narmada rift and Kathiawar-Kutch rift in a east-west direction. The Cambay region may therefore be considered to have a lithospheric expression of a mantle plume (Bose 1972). The dyke clusters of west coast, Panvel flexure and geothermic field of Konkan region indicate a wedge like mantle protruberance. This is also corroborated by the 50 km wide N-S trending elliptical gravity high (60 mgals) between Bombay and Surat.

The other important features of the gravity field of Indian shield are two
negative Bouger anomalies covering significantly large areas. These include negative anomalies −110 mgal and −90 mgal around Koyana–Kerad region in a N–S direction and around Kurudwadi in a NW–SE direction respectively. These gravity lows are likely to indicate deep infratrappean crustal faults whereas the gravity high indicates rise of Moho surface. The DSS profile along 17° and 18° latitudes provided by NGRI suggests a major mantle reaching sub-volcanic fault in the Archean lithosphere running N–S at Koyana and is suspected to be the chief cause of instability in this region.

Thus the Indian shield offers an unique volcano–tectonic set up in which the stupendous volcanic activity was favoured by continental rupture and plate motion across a hot-spot and plume generated rifts controlled emplacement of alkaline dyke clusters at the close of the main volcanic episode. In such an environment of rapid accretion and slower consumption of lithospheric plates, where large horizontal compressive stresses are expected to prevail, increased fluid pressure along the zones of maximum fracture porosity might have induced crustal failure and resultant seismicity.

8. DISCUSSION

There can be hardly any doubt about the fact that seismic activity of the Koyana region significantly correlates with reservoir level fluctuation at least for the period 1963–71. Highly contrasting seismicity around large reservoirs all over the world in diverse geological setting indicates that a suitable tectonic setting is also an essential prerequisite for resultant seismicity to be stimulated by extraneous reservoir loading. The stimulated seismicity is most likely to occur in areas of largely stressed normal or transcurrent faulting with extensional strain and least likely in areas of thrust faulting with contractional strain (Castle et al 1980).

The available data, although inadequate, on the structure and tectonics of Indian ocean and the shield area indicate that an unique volcano–tectonic set up exists in this region. The great volcanic province owes its existence to the continental splitting, dispersal and subsequent volcanic activity nourished by the northward movement of the Indian plate across a hot-spot. The four-arm rift junction near Gulf of Cambay represents a mantle plume which controlled the infratrappean lithospheric structure and emplacement of alkaline sub-volcanics and carbonatite bodies.

Continuous accretion of Indian ocean floor at a faster rate than the rate of plate consumption at the Himalayan subduction zone may cause upwarping of the water thin (very large length/thickness ratio) lithospheric plate by transmitting resultant great compressive forces for long distances through the rigid plate. In an analysis of a theoretical model of stress conditions of failure in fluid filled rocks, Balakrishna & Goud (1970) estimated the horizontal compressive stress component at the focal region in the Koyana area to be of the order of nearly 2,100 kg/cm². From the analysis of Koyana earthquake (main shock) accelerogram, it has been estimated that the regional tectonic stress field in the Koyana area is of the order of 140 bars (Guha et al 1974).

Therefore, Koyana region with high ambient stress field in a tectonic state other than thrust faulting identifies itself as potential for induced seismicity. Although there are examples of large reservoirs in such vulnerable zones, in various other parts of the world, without recognizable seismic activity, the data accumulated so far regarding the seismicity related to impoundment in the Koyana region at least establishes a tectonic probability for such phenomenon.

9. REFERENCES


ABSTRAIT

Dans le présent rapport sont envisagés les problèmes des études géologiques des massifs des roches carbonatées liés à la construction des ouvrages hydrauliques. On montre que la dissolubilité des carbonates qui détermine leur possibilité potentielle de karstification, depuis longtemps attire l'attention des ingénieurs-geologues en tant qu'un facteur le plus important qui définit la capacité de rétention d'eau des retenues. Actuellement, de grands succès sont obtenus dans l'étude des lois de développement du karst, de la mise au point des méthodes de son détection et de la réalisation des mesures de protection en URSS et dans d'autres pays. Dans les publications, sont largement présentées les particularités des roches carbonatées crayeuses, est déterminée la nature génétique de leurs propriétés spécifiques.

Moins nombreuses sont les publications concernant les données sur le comportement, dans les régions de leur développement, des roches carbonatées non-karstifiées pendant la construction des ouvrages hydrauliques à haute chute. De tels renseignements ne sont obtenus que le dernier temps grâce aux observations géophysiques, géotechniques et géodésiques de longue durée au cours de la construction et de l'exploitation des aménagements importants. Dans le rapport sont exposés certains résultats des observations de ce genre effectuées dans la zone de la retenue et du barrage d'Ingourï en Géorgie-ouest, qui ont mis en évidence dans les roches de fondation carbonatées quelques particularités spéciales de la dynamique du développement des processus de déformation, des variations de l'état de contrainte du massif et de la perméabilité de ce dernier, de la nature de transmission des charges sur la fondation du barrage, etc. L'auteur fait l'analyse des causes les plus probables des lois établies et indique les directions en perspective de l'étude des particularités du comportement des roches carbonatées saturées d'eau, soumises à des charges élevées.

ABSTRACT

The paper is concerned with the engineering geology study of carbonate rocks in relation to the construction of hydrodevelopments. It is shown that attention of engineering geologist has been for a long time drawn to the solubility of carbonate rock (which is a potential cause of its
...Les massifs des roches carbonatées servent souvent de fondation ou de milieu pour les ouvrages hydrauliques. L'expérience mondiale de la construction des barrages compte des centaines de grands ouvrages fondés sur des roches carbonatées. En URSS, par exemple, ce sont : barrage d'Ingouri sur l'Ingouri, de Tshirkei sur le Soulak, de Torktougl sur la Naryne, de Ladjanouri sur la Ladjanouri, de Pavlovsk sur l'Oufa, de Kakhovka sur le Dniepr, de Narva sur la Narva, de Tcherkav sur le Tchirchik, de Chaari sur la Didi-Tchala, de Volkov sur le Volkov, de Plavinas sur la Divna occidentale, etc. D'après les projets soviétiques, des barrages sont construits ou en construction : le barrage de Babga sur l'Euphrate en Syrie, les aménagements de Ras seb en Tunisie, de Haditha en Irak, de Hpa Binh au Viêt Nam, etc. (5).

C'est depuis longtemps déjà, que les roches carbonatées attirent l'attention des ingénieurs géologues. Cette attention est attirée, tout d'abord, par la dissolution des carbonates, qui détermine leur possibilité potentielle de karstification. La découverte du karst dans les appuis des barrages et les bords des retenues amenait souvent à des modifications fondamentales des projets, parfois même, au remembrement de ces derniers. La sous-estimation de l'influence du karst sur la capacité de retention d'eau des retenues ou l'insuffisance des renseignements peuvent provoquer des conséquences assez graves. A titre d'exemple, il suffit de rappeler de vaines tentatives de plusieurs années visant à remplir la retenue de l'aménagement de Dokan en Irak, où les dépenses pour les travaux d'injection, destinées à la lutte contre la filtration au large, sont comparables au prix du barrage même (11). De pareilles difficultés ont eu lieu pendant la réalisation de l'aménagement de Keban sur l'Euphrate en Turquie. C'est pourquoi, une grande attention a été toujours prête aux problèmes de l'étude du karst, y compris les études théoriques et expérimentales de sa formation, de sa situation spatiale, de son morphologie, de caractère des matériaux de remplissage, et les prévisions sur le développement des filtrations et des effets renard dans les massifs des roches karstifiées après le remplissage des retenues et la transformation de leurs rives. Notamment, en URSS, sont mises au point et appliquées avec succès les méthodes de l'étude des roches karstifiées pour la construction des ouvrages hydrauliques, nettement formulées par M. Lykochine (4). Elles peuvent être ramenées à :

1. Mise en évidence des zones de développement du karst par rapport aux formes érosives du relief, à la lithologie des roches et aux éléments structuraux et tectoniques. Il est très important, avec cela, de déceler la correspondance des différentes formes du karst

VII.178
(horizontales, verticales, d'effon- 
drement, etc) à de diverses profon- 
deurs, de mettre en évidence les 
zones de tal ou tal karst par rap- 
port aux talus et terrasses des val-
lées fluviales, aux thalwegs des 
ravins, etc.

2. Etude de l'histoire de la 
formation du relief actuel et an-
cien du terrain, de son réseau hyd-
rogaphique, avec l'analyse des 
etapes de son développement et des 
particulaties de son existence, 
auxquelles se rapportent: durée de 
telles ou telles conditions de la 
formation de l'écoulement de sur-
face et souterrain, alimentation 
et drainage des eaux souterraines, 
caractéristiques de l'accumulation 
des formations fluviales et superti-
cielles qui changeaient les condi-
tions d'infiltration des eaux météo-
riques et celles de drainage des 
eaux souterraines (alluvion, dolu-
vium, moraine, eluvium, etc).

3. Etude des dimensions et des 
formes des manifestations du karst, 
de leurs interdépendances tridimen-
sonnelles, des dépendances de la com-
potition lithologique des roches en-
caissantes, de leur fissuration, 
conditions de glissement, etc.

Une tendance, un peu différente, 
à été admise pour l'étude des roches 
carbonatées faiblement karstifiées, 
telles que: craie, calcaires craye-
xes, dolomites et marne. Actuelle-
ment, on a étudié avec soin la na-
ture génétique de la résistance et 
de la déformabilité des formations 
crayeuses, conditionnée par les par-
ticularités de leur composition mi-
neralogique et de leur microstruc-
ture (8). De telles caractéristiques 
geologiques importantes des roches, 
comme résistance au cisaillement, 
compressibilité sous chargé, imper-
meabilité, propriétés thixrotropes 
et rhéologiques, sont liées, le 
plus souvent, à la teneur en argile 
des dépôts ou caractère des con-
tacts entre les particules élémentai-
tes les composant, d'origine 
organogene, chimique et terrigène. 
Dans la plupart des cas, cette re-
lation porte un caractère fonction-
nel, et l'affaiblissement dans cer-
tains échantillons des liaisons de 
corrélation entre les paramètres 
envisages s'explique, généralement, 
par les défauts de la microstruc-
ture, tels que microfissuration, 
inclusions, etc. Notamment, la 
capacité de rétention d'eau des 
retoues, réalisées dans les régions 
du développement des roches craye-
uses (7), est déterminée par le 
pourcentage des particules argil-
ieuses dans les dépôts et par la 
teneur en dolomites des roches (ce 
phénomène est observé, d'habitude, 
dans les zones arides). C'est ainsi 
que les craies argileuses et les 
dolomites crayeuses ont des propriétés 
rhologiques marquées, et, par con-
seqüent, la capacité à la cicatrisa-
tion des fissures, qui les fait 
pratiquement imperméables. Les co-
efficients de perméabilité typiques 
dans le massif de ces roches consis-
tuent des centièmes et des millés-
mes de m/j. Avec la dolomitisation, 
les roches crayeuses perdent de 
ses propriétés rhologiques et 
deviennent plus fragiles. Cela se 
manifeste, partiellement, grâce à la 
"prise" par les cristaux de dolo-
mite de nouvelles formes, d'une 
partie considerable des particules 
craieuses qui quittent, ainsi, la 
sphère de la formation des struc-
tures, et, partiellement, à la 
suite d'une brusque augmentation de 
de la superficie des contacts des 
grains cimentés. Par conséquent, 
de dolomites crayeuses secondaires 
se caractérisent par une fissura-
tion plus grande et une perméabi-
lité correspondante. Les coeffici-
ents de perméabilité dans les zones 
d'accidents tectoniques peuvent 
atteindre des dizaines de mètres 
par jour. Par contre, les dolomites 
chemogènes crayeuses primaires, 
propres aux régions de sédimenta-
tion accumulative aride, surtout 
ses différences argileuses, ont, 
comme craie et marne, une petite 
perméabilité, mais aussi une faible 
résistance aux déformations de per-
meabilité qui apparaissent même aux 
faibles gradients de la charge le 
long des fissures.

La présence dans une même 
assise de couches des dolomites 
chemicals primaires et des calcaires 
dolomitiques réitérativement peut 
aboutir, dans les conditions hydro-
logiques favorables, à l'acti-
vation des processus karatiques, 
celles qui elles s'est produit dans 
l'appui rive gauche du barrage 
Tabqa sur l'Euphrate en Syrie,(1). 
Et avec cela, la particularité spé-
cifique du pareil karst consiste 
en remplissage, complet ou partiel.
des cavités formées par la "farine" dolomitique - produit du transport par débouchage, suivant les fissures, des particules fines de dolomites (g'habitude, des petits cristaux séparés, libérés pendant la dissolution et l'affaiblissement dans les roches des contacts entre les éléments précipités). Une propriété caractéristique de la "farine" dolomitique est sa perméabilité et sa capacité d'être soumise à des déformations de la perméabilité.

Jusqu'à présent on a prête peu d'attention à certains problèmes importants relatifs à l'étude des conditions de construction de grandes retenues dans les régions des roches carbonatées faiblement karstifiées. Cela se rapporte aussi à la dynamique du développement des processus de déformation, aux particularités des variations de l'état de contrainte du massif et à leurs caractéristiques de perméabilité, au caractère de transmission des charges sur la fondation du barrage (poisson d'Archimède), etc.

Actuellement, l'étude de ces problèmes compliqués devient possible grâce à l'organisation aux aménagements hydrauliques d'Ingouri et de Tektogoul des observations géophysiques et géotechniques à long terme et des observations géodesiques d'une précision élevée. Des données très intéressantes ont été obtenues, lors de ces observations, dans la région de la retenue et du barrage d'Ingouri. Ces données sont importantes à cause des caractéristiques exceptionnelles des ouvrages de l'aménagement et par suite de la présence à côté des calcaires et des dolomites, dans la partie aval de la retenue, d'autres différences lithologiques des roches ce qui permet de comparer les particularités du comportement des massifs, présentés par ces roches, sous action des effets statiques et dynamiques analogues.

Le barrage - voûte d'Ingouri, ex construction sur l'Ingouri en Géorgie ouest, a une hauteur de projet de 270 m et a créé une retenue de 1,1 km³ de capacité. Dans la fondation du barrage et de ses appuis, ainsi que dans la majorité de la partie aval de la retenue, se trouvent les calcaires, calcaires dolomitisés et dolomites du

barème dont les couches, avec un pendage général aval, forment dans l'axe de la vallée une petite casure anticlinale (5). Parmi les calcaires et les dolomites se rencontrent de rares et peu puissants bancs de marnes et des inclusions de chaînes.

En général, les roches carbonatées sont très fissurées. Les fissures fermées prédéterminent ; ce ne sont que 25 fissures de toutes les fissures enregistrées, qui ont l'ouverture dépassant 10 cm et l'orientation, dans la plupart des cas, suivant la stratification.

Les phénomènes karstiques dans les calcaires et les dolomites du site du barrage dans les limites des côtes du remouls sont faiblement développés, ce qui s'explique par le retardement des processus karstiques par rapport à l'approfondissement rapide de la base locale de l'érosion. Dans la majeure partie de la superficie des versants de la vallée d'Ingouri le karst porte un caractère embryonnaire, présent par des formes superficielles peu profondes, telles que petites fontaines séparées et canaux suivant les grandes fissures et les caps isolés des dolomites crayeuses. Il y a, aussi, un remaniement par débouchage du matériau des zones de broyage tectonique avec son transformation en 'farine' dolomitique. L'épaisseur de cette zone faiblement karstifiée ne dépasse pas 20 à 40 m.

L'analyse du changement de la fissuration des roches avec la profondeur, la composition et les propriétés du remplissage des fissures, ainsi que des caractéristiques élastiques des roches, décelées par les méthodes géophysiques, a permis de définir la zonale du massif d'après l'intensité d'altération et de décharge. On a établi que l'épaisseur de la zone d'altération n'est que de 5 à 15 m et celle de la zone de décharge intense latérale de 10 à 50 m.

Ainsi, les résultats d'études mentionnées permettent de voir que le rôle des processus exogène dans la formation des propriétés géologiques du massif entraîne à l'interaction active avec les ouvrages de l'aménagement d'Ingouri est assez limité. Dans la plupart des cas, les processus d'altération, avec
leur vitesse, sont ici en retard considérable par rapport à la dissolution et l'entraînement des carbonates. Il est vrai qu'un autre tableau se présente dans les dolomites, dont la dissolution est accompagnée souvent par la formation de la "farine" dolomitique. Néanmoins, en général, les fissures tectoniques et lithogénétiques dans les dépôts carbonates, même dans les limites de la zone d'alteration intense, sont, d'habitude, moins remplies du matériau dispersé, faiblement permeable, que dans d'autres roches. L'exception font les marmes, les calcaires argileux et les dolomites dont la composante insoluble peut s'accumuler dans les grandes fissures ou les cavités karstiques, dans les conditions hydromécaniques correspondantes.

Dans ce cas, les calcaires creux et les dolomites se caractérisent par une faible teneur en argile. C'est pourquoi, la partie éoréante du remplissage des fissures de la fondation et des appuis du barrage d'Ingoury est présentée par la "farine" dolomitique, très perméable. Il en résulte, la perméabilité relativement élevée du massif des roches carbonatées, ayant les absorptions d'eau spécifiques moyennes de 3 l/min (aux profondeurs de 10 à 30 m) à 0,1-0,5 l/min (aux profondeurs dépassant 50-75 m). Il est significatif que la zone supérieure (assez épaisse dans certains types de roches) de faible perméabilité grace au colmatage des fissures par des produits d'alteration, n'est pas observée dans le massif.

Un autre tableau est observé dans les dépôts volcanogènes - terrigenes du Jurassique, sous-jacents à l'assise carbonatée du barème et sortant à la surface à la proximité immédiate du barrage, dans les limites de la partie profonde de la cuvette de la retenue (greq, andésites, tufs, aleurolites, grès tubacés). Ils se caractérisent par une perméabilité très faible (prédominent les coefficients de perméabilité de dixièmes et centièmes de m/j), surtout, dans les parties du massif proches de la surface, malgré la fissuration presque aussi élevée que dans les roches carbonatées. Cela s'explique par le remplissage de la partie dominante des fissures par les produits argileux d'alteration.

Les études prolongées géophysiques, in situ, géodesiques et dihrométriques - déformographiques (9), réalisées dans la zone du barrage d'Ingoury et dans la partie aval de sa retenue, permettent de resoudre les problèmes suivants:
- nature et échelle du changement des propriétés et de l'état des roches dans la fondation du barrage pendant la construction, le remplissage et l'exploitation de la retenue;
- particularités de la déformation du massif dans la fondation du barrage-voûte et dans la partie aval de la retenue;
- causes et dynamique des séismes provoqués par les barrages;
- définition de l'influence exerçee à la sismicité et au régime sismique de la région par la création et l'exploitation de la retenue profonde du barrage d'Ingoury.

La solution de ces problèmes par les méthodes géophysiques s'effectue sur la base de l'étude des lois des variations dans l'espace et le temps du champ des vitesse et des paramètres d'affaiblissement des ondes élastiques de la gamme ultra-sonore, sismologique et de prospection sismique de fréquences, du champ des résistances électriques et du champ de radiation sismique.

Les premiers résultats d'études montrent déjà que dans la zone superficielle de la croute terrestre, au site du barrage-voûte et dans la partie aval de la retenue, le remplissage de cette dernière fait intensifier les différents processus de déformation qui provoquent de brusques variations des paramètres du champ des ondes sismiques sur le tronçon du profil géophysique spécial de fond (10). Le résultat le plus intéressant des observations sur le profil est la mise en évidence de la différence prononcée dans la nature des processus de déformation ayant lieu dans les blocs géostructuraux importants, constitués par les roches carbonatées et les dépôts volcanogènes-terrigenes. Les premiers se caractérisent par les abaissements stables de la vitesse des ondes élastiques en fonction nette des étapes du remplissage de la retenue.
Dans les deuxièmes, les abaissements de ce genre ne sont pratiquement pas observés a n'importe quelle échelle du niveau du relevage. Evidemment, cela témoigne de la différence considérable entre les processus de déformation qui se passent dans la cuvette et aux bords de la retenue, constituées des divers, complexes des roches. L'analyse détaillée des résultats des études montre que la cause principale de cette différence consiste en grande diversité de la perméabilité des massifs des roches carbonatées et celles tufogènes-térrigenes. Les roches carbonatées, très perméables, plusieurs fois sont plus sensibles aux variations de la valeur et du caractère de la pression interstitielle dans le massif, du au brusque changement du régime hydrologique de la région lors de la création et de l'exploitation de la retenue. Avec cela, l'action dilatante de l'eau est si forte qu'elle fait égaliser, pratiquement, les processus de compression des roches sous le poids de l'ouvrage et de l'eau dans la retenue en provoquant la décompression totale (poussée d'Archimède) du massif et l'élévation de certaines zones. Il est probable que l'élévation totale de la surface de la fondation et de l'amont du barrage après le remplissage de la retenue, peut être expliquée par l'influence de ce phénomène.

La compagne périodique annuelle des différents champs géophysiques, faiblement reconnaissable dans la région du développement des dépôts volcaniques-térrigenes, manifeste assez bien dans la partie superficielle du massif, dans les blocs structuraux présents par des roches carbonatées, et s'affaiblit d'ensemble avec la profondeur. Les études spéciales (2) ont montré que le rôle principal appartenait ici aux variations saisonnières de la température de l'air et aux variations du régime thermique du massif, liées à cette dernière. Le profondeur de la zone indiquée est déterminée par l'intensité de l'échange convectif de chaleur et, par conséquent, dépend fortement de la perméabilité de l'assise étudiée. On a constaté que dans les roches carbonatées de la région du barrage d'Ingoury, l'épaisseur de la zone d'échange actif de chaleur pouvait atteindre une profondeur de 80 à 90 m, où continuaient de se manifester l'intensité associative et les variations observées des paramètres géophysiques conditionnés par les contraintes thermiques alternées.

Les particularités envisagées du comportement des massifs fissurés des roches carbonatées saturées d'eau, sur lesquelles reposent les cuvettes des retenues profondes, ne constituent qu'une partie des problèmes étudiés au cours des observations prologées, réalisées à l'aménagement d'Ingoury. Des résultats importants ont été obtenus aussi pendant l'étude des fluctuations dans l'espace et le temps du champ de radiation sismique. Notamment, les observations ont mis en évidence une regularité certaine dans la naissance et la migration des ondes "induites", nettement, liées à la nature et à l'intensité des processus déformatifs dans le massif, provoqués par la mise en eau du barrage (3). Il y a encore des renseignements concernant la manifestation dans les massifs des roches carbonatées, associées à des charges élevées, des propriétés peu typiques aux rocher, comme propriétés rhéologiques. Cependant, ces processus et phénomences nécessitent encore des études approfondies ultérieures sans lesquelles sont impossibles, actuellement, leur interprétation géologique pure et, par conséquent, l'authenticité des prévisions des conditions et du caractère de leur développement.

En conclusion, il faut souligner encore une fois que les roches carbonatées ont joué et joueront toujours un rôle important comme fondations ou milieu pour les ouvrages hydrauliques, notamment, pour des retenues importantes. Les particularités spécifiques des massifs carbonatés associés aux roches, leur comportement aux différentes phases de la construction et de l'exploitation des ouvrages hydrauliques et hydroélectriques, le caractère d'interaction avec ces derniers, nécessitent une grande attention de la part des ingénieurs-géologues. En même temps, en donnant la priorité à l'approfondissement et au perfectionnement de nos connaissances sur la nature du développement des
processus karstiques, les méthodes de leurs études, il convient de continuer les investigations approfondies des autres propriétés caractéristiques des formations carbonatées, décelées seulement ce dernier temps grâce aux observations complexes de longue durée, effectuées sur les ouvrages en construction ou en service. Cela permet d'organiser les recherches avec plus de tendance, d'augmenter considérablement l'authenticité des prévisions géologiques et, par conséquent l'efficacité des mesures de protection projetées.

REFERENCES


Theme 5

ENGINEERING GEOLOGICAL PROBLEMS OF SEA-COAST AND SHELF AREAS
ABSTRACT

Engineering geologic investigations of the continental shelf and slope are needed to ensure safe and economical recovery of petroleum resources. There are two general categories of marine engineering geologic investigations: regional studies to determine general conditions over large areas, and site-specific studies for final design and siting of structures. Regional studies are discussed in this paper. They are done mostly in frontier areas to provide a basis for selecting exploratory drilling equipment, for feasibility studies, and for preliminary construction planning and cost estimating.

Regional mapping is based on high-resolution geophysical data and on soil samples. Typical regional surveys include collection of several hundred to several thousand kilometers of data along survey grid lines spaced 0.5-km or more apart; samples are collected at key locations based on review of the geophysical data. Geophysical survey equipment includes a water-depth recorder, a side-scan sonar to provide a plan view of the seafloor, and several sub-bottom profilers to show strata to depths of at least several hundred meters below the seafloor. Sampling is done with gravity-dart or piston corers.

Regional maps and cross sections are drawn at scales between 1:25,000 and 1:500,000 to show (1) water depth and seafloor topography, (2) thickness and distribution of major soil and rock units, and (3) geologic features of potential engineering significance, such as faults and submarine landslides. These basic illustrations are then used to develop derivative maps showing, for example, anchoring conditions, jack-up rig considerations, relative difficulty of pile driving and seafloor trenching, relative slope stability, and considerations for preliminary platform siting and pipeline routing.

ABSTRACT

Des investigations géologiques du plateau continental et du talus sont nécessaires pour assurer la récupération des ressources pétrolières d’une manière safe et économique. Il y a deux catégories d’investigation géologique sous-marine: la première est une étude régionale pour déterminer les conditions générales des surfaces répandues, et la seconde est une étude plus raffinée avec des plans finals pour l’emplacement des bâtisses. Dans cet article, l’auteur décrit les études régionales d’endroits nouveaux pour établir une base pour les plans conceptuels des structures et pour s’organiser d’une manière pratique.

La cartographie régionale est fondée sur les données géophysiques à haute-résolution et sur le prélèvement des carottes. Des relevés régionaux typiques consistent...
de données amassées sur des centaines et des milliers de kilomètres parcourus le long des lignes d'un réseau séparées d'un 0.5-km ou plus, et l'endroit de prélèvement des carottes dépend sur la conclusion des données géophysiques. L'équipement des études géophysiques consiste d'une sonde pour déterminer la profondeur de l'eau, un sondeur à ultra-sons qui donne le relief sous-marin, et bien d'autres instruments pour indiquer et enregistrer les couches de terre à des centaines de mètres sous le fond de la mer. Quant aux carottes, elles sont préllevées avec des carottiers spéciaux.

Les cartes régionales et les sections transversales sont à l'échelle de 1:25,000 à 1:500,000 pour montrer (1) la profondeur de l'eau et le relief sous-marin, (2) l'épaisseur et la distribution des groupes de sols et de roches, et (3) les failles ou les éboulements sous-marins. Ces cartes régionales servent à faire d'autres cartes plus détaillées pour indiquer les manières d'ancrer, les difficultés relatives à la construction des tranchées au fond de la mer, à stabilité des pentes, et l'adaptation relative à l'installation des plates-formes ou pour la direction des pipe-lines sous-marins.

INTRODUCTION

Engineering geologic investigations are becoming increasingly important to exploration and development of offshore petroleum resources. This is because: 1) the effect that surficial geologic conditions can have on offshore activities is now widely recognized, and 2) the investment required for offshore exploration and development is very large. For example, the operating cost for a single exploration-drilling rig can be $100,000 or more per day; costs for the design and construction of deep-water production platforms are in the hundreds of millions of dollars. Unanticipated geological conditions can interfere with anchoring of drilling rigs and delay drilling operations; designing platforms without timely assessment of seafloor stability can result in extensive design changes and costly delays.

There are two general categories of marine engineering-geologic investigations: regional studies to determine general conditions over large areas, and site-specific studies for final design and siting of structures. This paper gives an overview of the methods and applications of regional engineering geologic mapping. The first section of the paper discusses the purpose and scope of regional mapping. This is followed by a description of techniques for field-data acquisition, basic mapping, and mapping for specific activities related to petroleum exploration or development. Finally, concluding comments are given.

PURPOSE AND SCOPE OF REGIONAL MAPPING

Mapping of large prospective areas, tens to hundreds or even thousands of square kilometers in extent, is done to provide an overview of geologic and soil conditions. This overview is critical to the efficient and economical development of frontier areas, for which there generally is little or no reliable information available. Information provided by regional mapping is used for numerous petroleum exploration and development activities including 1) selecting appropriate exploratory drilling rigs, 2) predicting geotechnical problems related to exploratory drilling, 3) conceptual design of foundations, 4) preliminary selection of platform sites and pipeline routes, and 5) preliminary construction planning and cost estimating.

Regional maps are also useful for planning adequate site-specific studies for final siting and design. Without the benefit of regional maps, areas investigated during site-specific studies are sometimes too small to allow reliable assessment of geologic conditions. This is especially true when sites are in large areas of potential seafloor instability (Floessel and Campbell, 1980).

Regional mapping, like most engineering geologic mapping of offshore areas, is based on interpretation of high-resolution geophysical data and on seafloor samples. Thus, the first phase of a regional investigation is to conduct a geophysical survey and collect samples. Geophysical data are collected along gridlines covering the area of interest. Next, based on review of the geophysical data, seafloor samples are collected at optimum locations. Data and samples are then taken to onshore facilities where data interpretation and sample testing are done. Lastly, basic interpretive maps and cross sections and maps for specific applications are prepared. Basic interpretive maps include bathymetric maps, materials maps, and geologic features maps. Maps showing anchoring conditions, pile drivability, and considerations for pipeline
routing or platform siting are examples of derivative maps prepared for specific applications.

GEOPHYSICAL SURVEYING AND SEAFLOOR SAMPLING

Geophysical Equipment

Several specialized geophysical systems are operated simultaneously along the survey gridlines. These systems provide information about water depth and seafloor topography, soil strata to depths of at least several hundred meters below the seafloor, and geologic conditions. Each of the systems operates by transmitting acoustic pulses and displaying the reflections, from the seafloor or buried strata, on a continuous graphic record. Although there are many trade names for the various systems available, the three basic categories of systems used for regional studies are described in the following paragraphs.

Water-Depth Recorder. A water-depth recorder is operated to provide accurate water depths. Normally, all reflections received by a water-depth recorder are from the seafloor directly beneath the survey vessel. Reflections are displayed as a function of time and are plotted side-by-side to provide a continuous profile of the seafloor. Water depth is determined based on the velocity of sound in water.

Side-Scan Sonar. The side-scan sonar provides a plan view of the seafloor (Fig. 1). The side-scan sonar system transmits a fan-shaped beam directed to either side of the survey vessel. All reflections are from the seafloor but, unlike the water-depth recorder, reflections are from a strip of seafloor on either side of the vessel, rather than from directly beneath the vessel. As a result, side-scan records are similar to low-oblique aerial photographs. The width of the swath displayed on the record is variable, but is typically several hundred meters. Side-scan sonar records are useful for mapping areas of rock outcrop (Fig. 1), boulders, and other similar seafloor features.

Sub-Bottom Profilers. Sub-bottom profilers provide vertical profiles of strata beneath the survey vessel. These profiles are similar to a detailed cross-section (Fig. 2). Sub-bottom profilers operate at lower frequencies than most water-depth and side-scan systems, allowing much of the transmitted energy to penetrate the seafloor. Some of the energy is reflected from the top of each successively deeper soil or rock stratum.

Each of these sets of reflections is recorded as a function of time; successive sets of reflections are plotted side-by-side to form a continuous profile.

Three separate sub-bottom profiler systems, operating at different frequencies and providing different resolution, are normally operated simultaneously. Three systems are needed to provide the necessary detail in the shallowest sediments as well as information to depths of a few hundred meters. Shallow-penetration profilers portray strata to maximum depths of about 30 meters and maximum operational resolution is typically 0.5 to 1 meter. An example of a shallow-penetration profiler is a 3.5 kilohertz profiling system. Intermediate-penetration profilers portray strata to maximum depths of 75 to 200 meters, depending on the specific system being used (Figs. 2, 3, and 4); maximum operational resolution is typically 1 to 2 meters. Acoustipulse, other boomer-type systems, and minisparkers are examples of intermediate-penetration profilers. Deep-penetration profilers portray strata to maximum depths of about 900 meters and maximum operational resolution is about 9 meters (Figs. 5 and 6). Examples include water guns, air guns, sparkers, and minisleeve exploders. Actual penetration for all systems is largely a function of the acoustic characteristics of the soil or rock and the frequency of the system. For example, the greatest penetration is achieved in soft clayey sediments, and little or none in hard rock (Fig. 7). Lower frequency systems provide greater penetration, but have less resolution.

Survey Grid

Grids for most regional surveys cover at least several hundred square kilometers and consist of two sets of parallel lines intersecting at right angles. Line spacing depends on geologic and operational considerations but is as much as several kilometers or more. A representative grid might consist of dip lines at intervals of 0.5 or 1-kilometer and strike lines 2 to 5 kilometers apart.

Survey Vessel and Navigation

Regional surveys are usually done with a vessel between 30 and 80 meters in length and require a scientific crew of 7 to 10 people. Underway navigation is provided by any one of a number of commonly available radio-positioning systems. These systems are sometimes used with a satellite navigation backup system, especially in remote areas. As the vessel travels along the survey gridlines, posi-
Fig. 1: Side-scan sonar record showing rock outcrop, Gulf of Alaska.

Fig. 2: Intermediate-penetration profiler (Acoustipulse) record showing angular unconformity between Holocene silty clays and dipping Tertiary strata, Gulf of Alaska.
Fig. 3: Intermediate-penetration profiler (Acoustripulse) record showing buried fault, deformed strata, and shallow gas, Santa Barbara Channel, California.

Fig. 4: Intermediate-penetration profiler (Acoustripulse) record showing two modern mudflow deposits overlying undisturbed marine sediments off Mississippi Delta.
Fig. 5: Deep-penetration profiler (sparker) record showing fault, fault scarp, and folded strata, Bering Sea, Alaska. Inferred moderately overconsolidated sediments are southwest of fault and normally consolidated sediments are to northeast. See Figs. 10 and 11 for location of record.

Fig. 6: Deep-penetration profiler (sparker) record showing buried landslide deposits, Gulf of Mexico. Sliding occurred during the late Pleistocene.
Fig. 7: Intermediate-penetration profiler (Acoustipulse) record showing major fault, Gulf of Alaska. Rock is northwest of fault and about 40 meters of silty clay overlying glacial deposits is to southeast.

fixmarks are made at intervals of 200 to 500 meters, depending on the particular survey. Absolute accuracy of fixmarks along gridlines is within 50 meters and usually within 20 meters.

Seafloor Sampling

Seafloor sampling is done from the geophysical survey vessel at locations selected from review of the geophysical data. Sampling stations are chosen so that the range in geotechnical properties of surficial soils can be defined with the minimum number of samples. The sampling systems used for regional studies range from simple grab samplers to large gravity drop-core or piston samplers weighing up to 1000 kilograms. These large samplers provide good-quality cores up to several meters in length and 7 to 10 centimeters in diameter. Less commonly, soil borings are done as part of regional studies to determine the geotechnical properties of soils to depths of 100 to 200 meters. For seafloor sampling programs, visual descriptions of samples and simple testing, such as with a Torvane, hand-held penetrometer, or miniature vane, are done on the survey vessel. Additional classification and strength testing is usually done in the onshore laboratory.

Length of Survey

Regional surveying and sampling is normally done 24 hours per day. Usually, 100 to 150 kilometers of gridline can be surveyed per day. Time for sample collection is a function of distance between sampling stations and water depth. Where water depth is less than 300 meters, the entire sampling operation at each location requires 15 to 30 minutes. Most regional surveys require collection of 1000 to 5000 line-kilometers of geophysical data and up to several hundred seafloor samples. Thus, overall time required to complete a regional survey normally takes several months when weather, equipment, and positioning downtime are considered along with port calls for re-provisioning, re-fueling, and crew rotation.

Additional information about marine geophysical and sampling techniques for engineering geologic investigation of offshore areas is given by McQuillen and Ardus (1977), Sullivan (1980), and Van Overeem (1978). Special operational considerations for surveys where water depths are greater than 200 meters are discussed by Campbell and others (1982).

BASIC INTERPRETIVE MAPS

Several types of basic interpretive maps and cross sections defining bathymetry, earth materials, and shallow geologic conditions are prepared. These illustrations are based on a synthesis of
all high-resolution geophysical data and soil sample information. These generalized maps and cross sections are usually at a scale of 1:25,000 to 1:500,000, depending on the size and geologic complexity of the area. Basic mapping can take two to three times longer than field data acquisition.

**Bathymetric Map**

The bathymetric map shows water depth and seafloor topography, and is based mostly on profiles from a water-depth recorder. Contour intervals vary from a few to tens of meters, depending on the relief and degree of seafloor irregularity. Shading is used to indicate areas where topography is too intricate to contour reliably. Side-scan sonar data may be used to help map the extent of irregular topography.

**Earth Materials Maps**

Extensive experience has shown that major soil and rock units generally correlate with individual acoustic units displayed on sub-bottom profiles. An example of good correlation between soil strata encountered in a boring and acoustic units is shown on Fig. 8. This general correlation between acoustic and materials units allows major soil and rock units to be mapped from geophysical data. During regional studies, geotechnical properties of the surficial soil (acoustic) units can be determined from seafloor samples. For deeper, buried units not sampled, the general nature of the materials can often be inferred from their acoustical properties coupled with a knowledge of the geologic setting (Fig. 9). These inferences, although qualitative and subjective, can provide reliable reconnaissance information when made by an experienced interpreter. A detailed discussion of techniques used to predict materials from sub-bottom profiles is in Randall (1980).

As discussed in the following paragraphs, geophysical data and sample information are used to prepare several types of regional maps showing the general distribution of soil and rock units to as much as 200 meters below the seafloor. Because information collected during regional studies is not used for final design, detailed geotechnical information is neither necessary nor warranted. The purpose of these regional maps is to give an overview of materials rather than detailed information for specific sites.

**Seafloor Materials Map.** A seafloor materials map shows the distribution of surficial soil and rock. This map is based on a combination of seafloor samples, and side-scan sonar and sub-bottom profiler data. The extent of any rock outcrop is mapped using side-scan sonar (Fig. 1) and sub-bottom profiler data (Fig. 7). Sub-bottom profiles are also used to map the extent of surficial soil units where units are greater than about 1 meter thick. Sometimes, major changes in texture of the seafloor soils can be mapped using side-scan sonar records.

**Subsoils Map.** When buried materials appear to be variable or where there is an erosional unconformity at the base of the surficial soils, a "subsoils" or "subcrop" map may be prepared (Fig. 10). Note: to facilitate comparison of the several types of maps discussed in this and later sections, example maps included as Figs. 10-14 are maps of the same area. These maps were originally prepared as part of a regional study of the southern Bering Sea. For regional studies, the subsoils map is commonly based only on sub-bottom profiles as samples from below the surficial materials usually are not available. As discussed previously, the general nature of buried soils or rock often can be mapped based on acoustic character.

Some subsoil maps portray the relative degree of overconsolidation and the distribution of normally consolidated subsoils and rock (Fig. 10), where subsoil strata have been deformed and partly eroded, the degree of overconsolidation often is in direct proportion to the amount of erosion. For example, the degree of overconsolidation in the dipping Tertiary strata in Figure 2 increases from west to east. In these cases, the amount of erosion is reflected by the shallow geologic structure, and mapping the relative degree of overconsolidation is based partly on structural contours drawn on shallow horizons. For example, the greatest degree of overconsolidation would normally be along the crest of partly eroded anticlines, whereas less overconsolidation would normally be along the axis of adjacent synclines. Sharp contrast in the consolidation state is sometimes found across faults (compare Figs. 5 and 10). Thus, shallow structure maps sometimes are useful tools for predicting regional overconsolidation trends.

**Soil Similarity Map.** Where the degree of overconsolidation or the general type of materials cannot be predicted, a map showing the relative similarity of materials is sometimes prepared. Map categories are defined based on the overall acoustic character and the shallow geologic structure to depths of about 200 meters. The intent is to map categories
Fig. 8: Intermediate-penetration profiler (Acoustipulse) record showing correlation between acoustic units and soil strata at soil boring, Gulf of Mexico.

Fig. 9: Intermediate-penetration profiler (Acoustipulse) record showing inferred soil and rock strata, Lower Cook Inlet, Alaska.
Fig. 10: Subsoils map, portion of Bering Sea, Alaska. A veneer of soft clay, 3 to 6 meters thick, overlies most of area but is not shown. Same area shown on Figs. 11, 12, 13, and 14. Sparker record is on Fig. 5.

Fig. 11: Geologic features map, portion of Bering Sea, Alaska. Same area shown on Figs. 10, 12, 13, and 14. Sparker record is on Fig. 5.
such that each category consists of materials with grossly similar geotechnical properties. However, the actual properties of materials comprising each category are not determined until later, based on soil borings. This map provides a rational basis for locating the least number of borings needed to define the general range in geotechnical properties of materials underlying large, regional areas.

Isopach Map. Isopach maps of surficial sediments are prepared when the surficial material is geotechnically distinct from the underlying unit and is thick enough to be discerned on sub-bottom profile s. Thickness variations are shown by contours whenever possible. Rarely, isopach maps of buried soil units may be prepared. Other maps showing the depth to the top of significant units, such as granular layers or rock, are also prepared whenever feasible.

Geologic Features Map

Geologic features of potential engineering significance are mapped based on all available data (Fig. 11). Some features are identified and mapped from only one type of data; others are mapped from a combination of data from two or more geophysical systems. Some of the more important geologic features mapped during regional studies are discussed in the following paragraphs.

Areas affected by mass movement.
Landslides (Fig. 6), slumps, mudflows (Fig. 4), and other types of failures are found offshore. Large failures can be tens to hundreds of square kilometers in extent and 150 meters or more thick. Modern mudflows off the mouth of the Mississippi River are well documented (Coleman and others, 1980). Costly mudflow-resistant platforms have been constructed in parts of this area where future mudflows are likely to cause large lateral loading on foundation members. Seafloor stability is usually the single most important engineering-geologic consideration for areas on the continental slope and off many deltas. To facilitate assessment of stability, modern and ancient failures are differentiated on features maps whenever possible. Various aspects of submarine slope stability are discussed in Richards (1977). Kraft and others (1979) discuss assessment of some types of seafloor instability.

Faults. Faults found offshore result from regional tectonic activity (Figs. 3, 5, and 7), from local tectonics (as in the vicinity of salt domes), and from differential settlement and deformation of soft sediments. All three types of faults can result in seafloor offset. Offset on faults caused by regional tectonics often occurs suddenly during an earthquake and is accompanied by ground shaking. Slow, aseismic creep offset is characteristic of most faults resulting from differential settlement (Kraft and others, 1979). Buried faults and faults with seafloor offset are differentiated on features maps to help assess the activity of individual faults and the seismicity of a region. Another important consideration for faults is that foundation conditions for some areas are significantly different on opposite sides of a fault where different materials have been juxtaposed (Figs. 3, 5, and 7).

Sand Waves. Sand waves or other bedforms may indicate modern transport of seafloor sediments and a high potential for scour around seafloor structures. Sand waves can be more than 20 meters high and 100 meters or more from crest to crest. Large sand waves are found in several areas, including Lower Cook Inlet in Alaska, on Georges Bank off the East Coast of North America, in the North Sea, and in the Straits of Magellan. However, sand waves in some areas are now inactive features that formed when sea level was lower and seafloor currents were higher than today. Sand waves in some other areas migrate only during storms.

Shallow Gas. Gas within a few hundred meters below the seafloor is common in some offshore areas (Fig. 3). Some shallow gas is generated in-situ from the decay of organic matter. Other gas forms at greater depths and migrates upwards into the shallow sediments. If gassy sediments are penetrated unexpectedly during either oil boring or petroleum drilling operations, a blow-out could result.

In some areas, small blow-outs probably occur naturally. Natural blow-outs apparently are caused by spontaneous degassing of near-surface, gas-charged sediments. Seafloor craters, commonly a few to a few tens of meters across and having up to 2 meters or more of relief, are sometimes associated with gassy sediments and probably resulted from degassing. Small mud mounds are found in some areas, especially in parts of the northern Gulf of Mexico, and may also be caused by gas expulsion. Shallow gas can also reduce the shear strength of associated sediments and, in extreme cases, affect foundation design.

Buried Channels. Buried channels are found beneath the continental shelf in some areas. Channels are most common within about 150 meters of the present sea...
surface and formed when rivers flowed across the continental shelves during sea-level lowstands. Channel-fill deposits can be more than 30 meters thick, but are usually thinner. Channel-fill deposits may have very different geotechnical properties than nearby non-channel deposits and may be characterized by rapid lateral and vertical variations.

Other features. Some conditions shown on the bathymetric or materials maps, such as slope gradients, areas of irregular seafloor topography, and unusual soil conditions, are compiled on the geologic features map whenever graphically feasible. Bathymetric contours may also be faintly printed on the features map. This results in a single, composite features map showing the relationships among the most important bathymetric, soil and rock, and geological conditions.

Cross Sections

In addition to the basic maps, several interpretive cross sections are prepared to show vertical relationships among soil and rock units and geologic features.

More information about regional engineering–geologic mapping of offshore areas or offshore geologic features can be found in Campbell and Hough (in press), Fanning (1980), Gunhilde and Roksengen (1980), King (1980), and Floressel and Campbell (1980).

APPLICATIONS MAPS

Applications maps show the engineering implications of bathymetric, soils, and geologic conditions for various petroleum exploration and development activities. These "derivative" maps are based on a general engineering assessment and synthesis of pertinent information shown on the basic interpretive maps. Applications maps are prepared for exploitation drilling, for feasibility studies, and for construction planning and cost estimating.

Exploration Drilling

Most offshore petroleum exploration drilling is done from either floating drillships or semisubmersibles, or from bottom-founded jack-up rigs. Floating rigs are usually anchored, but dynamic positioning is used at deepwater sites. A seafloor guide base is installed at all drilling sites. The guide base provides support for and guides the conductor pipe, casing, and drill string. The conductor pipe is the outermost casing and usually extends 50 to several hundred meters below the seafloor, depending on soil conditions. Maps may be constructed for the following activities/applications.

Drilling–Rig Selection. A rig selection map shows where water depth and soil conditions would allow use of jack-up rigs, where use of floaters with anchors is feasible, and where the water depth is so great that a dynamically-positioned rig would be required.

Anchoring. Anchoring maps show where soil conditions are suitable for using mud anchors or sand anchors, and where anchoring problems, such as limited penetration or unexpected breakout, can be expected (Fig. 12).

Jack-up–Rig Considerations. Jack-up–rig considerations maps show areas where penetration of spud-can footings may be excessive, where footing recovery may be difficult, or where unexpected punch-through and rig tipping may occur. Maps also show areas where mat-type footings would be appropriate and where seafloor scouring or mass movement may affect stability of a jack-up.

Guide Base Considerations. Maps for guide base considerations show areas of weak surficial soils where excessive penetration of guide bases could occur and areas where uneven support or tipping may result from rough seafloor topography, steep slopes, or seafloor scouring.

Conductor–Pipe Installation. Maps may show areas where boulders or rock will make installation difficult, or may merely show relative difficulty of conductor-pipe installation across the area.

Feasibility Studies

Various feasibility studies are done for large frontier areas to determine the economics of producing and transporting petroleum. Feasibility studies include selecting preliminary sites or pipeline routes and conceptual design of production structures.

Site Selection and Pipeline Routing. Generalized maps showing the relative favorability of areas for siting or routing are constructed to facilitate feasibility studies. Sometimes maps showing relative seafloor stability are made prior to preparing relative favorability maps. Relative stability is judged by considering slope gradient, nature of soil and rock, evidence for past or incipient failures, and any other pertinent factors. Maps prepared for preliminary design studies (Fig. 13) or for evaluating construction activities (Fig. 14) may also be used for preliminary siting and routing.

Conceptual Design. Figure 13 is an
Fig. 12: Map showing anchoring conditions, portion of Bering Sea, Alaska. Same area shown on Figs. 10, 11, 13, and 14.

Fig. 13: Map showing considerations for pile foundations, portion of Bering Sea, Alaska. Same area shown on Figs. 10, 11, 12, and 14.
example of a map showing considerations for pile foundations. This type of map is used when deciding the feasibility of using pile foundations (conceptual design) and for preliminary pile design.

Construction Planning and Cost Estimating
Maps showing difficult conditions or considerations for specific construction activities are used for 1) preliminary selection of construction techniques and equipment, 2) preliminary construction planning, and 3) for making preliminary estimates of construction costs. Pile driving and trenching are two common construction activities related to development of offshore petroleum.

Pile Driving. Maps showing the difficulty of pile driving are made based on the inferred nature of materials to a depth of about 150 meters. Some maps show only the relative difficulty of pile driving. For example, driving might be mapped as most difficult in overconsolidated soils along the crest of a partly eroded anticline and as least difficult in areas of undeformed, late Pleistocene and Holocene clays. Alternatively, a qualitative assessment of absolute driving difficulty is made and maps show, for example, areas where normal, moderately hard, or hard driving are anticipated (Fig. 13). If possible, contour maps showing the anticipated depth to refusal, on the top of rock or other hard stratum, may be made.

Trenching. Trenching maps usually show either the relative difficulty of trenching or a qualitative assessment of trenching considerations to depths of about 6 meters (Fig. 14). Special considerations include materials requiring blasting or other supplemental trenching techniques, high-plasticity clays (which can clog trenching equipment), and trench-wall stability.

CONCLUDING COMMENTS
The importance of regional engineering-geologic mapping of offshore areas is still not as widely recognized as is the need for site-specific engineering-geologic investigations. Nonetheless, regional mapping has been done for areas off North America, in the Bay of Campeche, off Venezuela, in the North Sea, off West Africa, and for some other areas where petroleum prospects are good. Virtually all of this work has been done in the past ten years, and most of it within the last five. The amount of regional engineering geologic mapping done in future years is expected to slowly increase as more and
more engineers come to recognize the considerable value of regional information. Recent development of deep-water sites on the continental slope, where geologic conditions are poorly known and the petroleum industry has little experience, has helped increase industry awareness of the need for regional mapping.

The reliability and usefulness of regional engineering-geologic maps should increase as more is learned about offshore geologic processes in general and as experience is gained in new frontier areas. The single most significant improvement in offshore engineering-geologic mapping during the next twenty years is anticipated to be development of a reliable method for direct, semi-quantitative identification of materials from seismic-reflection data. In other anticipated developments, marine engineering geologists will expand their scope of activity by applying regional mapping techniques to marine mining and other resource development projects.

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ETUDE DES PHENOMENES D'EROSION COTIERE LIES A L'EXTRACTION DE MATERIAUX SUR LE PLATEAU CONTINENTAL

STUDY OF COASTAL EROSION WITH THE EXTRACTION OF MATERIALS ON CONTINENTAL SHELF

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ABSTRACT

L'approvisionnement en matériaux (sables, graviers, minerais) nécessite des dragages importants sur les plateaux continentaux avec pour conséquences l'apparition de fosses profondes de quelques mètres et d'une étendue variable.

Ces excavations réalisées vont piéger les sédiments, et avoir des influences à plus ou moins long terme sur l'évolution de la morphologie côtière.

Il est donc nécessaire d'étudier ces évolutions pour interpréter leurs réelles conséquences et donc pouvoir orienter les futures zones d'extraction afin d'en éliminer les effets nuisibles.

Les techniques indispensables à l'étude de ces évolutions morphologiques sont la bathyétrie détaillée, le sonar latéral, la sismique haute définition, ainsi que les carottages pour caractériser les sédiments et connaître leur provenance.

En effet, les zones d'extraction peuvent soit piéger les sables côtiers qui migrent sous l'action de fortes houles et alors éroder les plages, soit piéger des sables du transport littoral et de ce fait diminuer l'alimentation de secteurs en voie d'accumulation ou en état d'équilibre.

Les différentes études doivent permettre de choisir les profondeurs et l'implanta-
tion de ces zones afin de remédier aux phénomènes nuisibles à la stabilité du littoral.

Dans cet article, le cas particulier d'un secteur d'extraction situé au large de l'estuaire de la Seine sera présenté.

ABSTRACT

The supply of materials (sands, gravels, ores) requires important dredgings on the continental shelves, from which the creation of deep pits of few meters and variable area results.

Such diggings trap the sediments and then will influence the evolution of coastal morphology on a more or less long period.

So the study of these evolutions appears to be necessary to evaluate their real consequences and then to allow us to choose the future extraction areas in order to eliminate bad effects.

The techniques required for the study of these morphological evolutions are: detailed bathymetric side scan sonar, seismic high resolution, as well as corings to characterize the sediments and determine their origin.

As a matter of fact, the extraction areas may either trap coastal sands moving under heavy swell action and then eat away the beaches, either trap sands of coastal transit and thereby reduce the supply of some areas in accumulating process or equili-
brating.

Several studies must allow us to choose the depths and the localization of these areas in order to remedy the effects prejudicial to the stability of the coastline.

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In this paper, we will make a study of an extraction zone which is located off Seine estuary.

I- INTRODUCTION

La demande en granulats utilisés pour les grands travaux (autoroutes, ports, etc.) et pour la construction, croît rapidement au taux de 11% par an alors que la demande annuelle est actuellement en France supérieure à 5 tonnes par habitant.

Parmi ces granulats qui sont soit d'origine naturelle (sables et graviers alluvionnaires, roches massives, concassées et broyées) ou artificielle (laitiers des hauts fourneaux), les sables et graviers représentent plus de 75% des tonnages produits. Jusqu'à ce jour, 160 km² du territoire national ont été exploités en carrière. Au rythme actuel, en 1985, c'est près de 750 km² qui seront exploités. Si cela peut paraître peu en regard de la superficie du territoire national, il faut savoir que les transports ont une incidence très forte sur le coût de ces matériaux, aussi les exploitations dans des sites proches des grands centres industriels et de ce fait le marché des granulats est régional.

Actuellement, ce sont donc les vallées alluvionnaires situées près des grands centres industriels et des zones urbanisées qui sont le plus activement exploitées. Or, leurs réserves ne sont pas inépuisables; les contraintes d'urbanisme, la protection des sites et le souci primordial de sauvegarder les nappes d'eaux souterraines tendent à stériliser une partie, chaque année plus importante, dans les zones où la demande se fait la plus forte. Aussi convient-il d'entreprendre, dès maintenant, la recherche de nouvelles sources de matériaux, en particulier sur les plateaux continentaux comme cela se pratique couramment en Grande-Bretagne et aux Pays-Bas.

Le dragage du fond de la mer entraîne certaines modifications temporaires ou permanentes du milieu marin. Ces modifications et leurs conséquences doivent être déterminées car elles sont susceptibles directement ou indirectement d'avoir des répercussions, en particulier sur la pêche, à la suite des changements qui peuvent en résulté dans les équilibres biologiques existant dans une région donnée. Dans certains cas les conséquences peuvent être néfastes ; dans d'autres cas, elles ne sont pas dommageables; elles pourraient même être bénéfiques. Tout dépend des espèces qui sont concernées.

La décision de mettre en exploitation une carrière sous-marine ne doit donc être prise qu'après avoir pesé les répercussions de cette activité sur les autres activités existantes ou envisageables dans le secteur maritime considéré.

Pour prendre cette décision avec le moindre risque, il faut :
- savoir définir dans la région considérée les conséquences probables de l'activité avant le démarrage de l'exploitation
- être capable de mesurer et de contrôler avec des moyens raisonnablement limités les effets du dragage durant l'exploitation

II- MODALITES DE L'INFLUENCE DU DRAGAGE SUR LE MILIEU MARIN

Les différents processus par lesquels le dragage est susceptible d'affecter le milieu marin ont fait l'objet d'inventaires tant en France qu'à l'étranger. De nombreux mécanismes ont été invoqués dont certains sont plus théoriques que réels.

On peut en résumer l'essentiel de la façon suivante :
(Figure 1 et tableau 1)
- La drague en modifiant la topographie du fond peut rendre la zone d'extraction impropre au charutage, même après cessation des activités. Si le dragage a lieu à faible distance des côtes, l'équilibre du littoral peut être affecté, entraînant des modifications des lignes de rivage et en particulier des plages.
- En cours de dragage, le pompage des matériaux au fond, de même que la surverse entraînent la mise en suspension d'une partie de la fraction fine du sédiment. Cette turbidité, indépendamment de la gêne intrinsèque qu'elle peut causer dans les zones touristiques, peut provoquer une baisse de la transparence de l'eau ayant pour conséquences une diminution de la pénétration de la lumière, donc de la photosynthèse. On a aussi évoqué le danger que puisse constituer la désorption d'éléments toxiques fixés par la vase au cours de sa remise en suspension. On a signalé enfin des sédimentations intemporelles dans les régions qui aboissonnent la zone d'extraction.
- Avec les sédiments, l'élinde pompe les organismes vivants sur le fond. Un certain temps est donc nécessaire avant qu'ils ne reviennent coloniser la zone qui a été exploitée. Parfois, même, si le substrat naturel de ces organismes à été levé, ils ne reviendront pas. Les cas sont à considérer : ou bien ces organismes présentent un intérêt en eux-mêmes (fraîères de harengs, coquillages comestibles) ou bien, sans avoir d'intérêt commercial,
ces organismes servent de nourriture à d'autres espèces activement exploitées (mâillon de la chaîne alimentaire).

- On discutera ces différents processus en s'appuyant soit sur l'expérience acquise en Baie de Seine, soit sur les données scientifiques disponibles.

Mais l'objet de cet article reste les effets au niveau du littoral.

Sur le plan morphologique, les gisements de matériaux du fond de la mer sont de deux types : ou bien ils consistent des reliefs (bancs et dunes sous-marines) dominant la topographie générale comme les accumulations de sables calcaires en Bretagne ou les "Bassurelles" de la Manche, ou bien, il s'agit du cours sous-marin d'anciennes fleuves, remplissage de chenaux fluviatiles (lits sous-marins de la Seine, de la Somme, de la Loire, etc...).

Si la zone exploitée est un relief, sa suppression ne gênera que le chalutage, à condition toutefois qu'une certaine quantité de matériaux soit laissée en place de façon à ne pas mettre à jour des aspérités éventuelles du substratum rocheux.

Si, par contre, on exploite des remplissages, chenaux, les excavations ainsi creusées peuvent constituer un obstacle au chalutage. On aura alors intérêt à ne pas laisser les extractions se faire de façon anarchique et extensive. Elles devront être réalisées en des sites bien définis, bien repérés et de superficies limitées sur les gisements de grosse épaisseur. L'exploitant disposeras ainsi d'un tonnage élevé sans avoir à modifier le fond de la mer sur une grande surface.

D'une façon générale, on devra éviter les exploitations extensives de minces couches de galets et de graviers par drague aspiratrice en marche qui laissent le fond de la mer labouré de sillons importants. Ces sillons ne disparaîtront pas nécessairement une fois l'extraction terminée et peuvent constituer un obstacle sérieux au chalutage.

Dans certains cas, le transit sédimentaire lié aux courants de marée peut amener, en quelques années, le comblement ou l'effacement des sillons. Mais il ne faut pas compter, à priori, sur un tel processus qui est loin d'être général. Une telle hypothèse doit s'appuyer sur une bonne étude des courants locaux et des transits sédimentaires qui y sont associés. Cette surveillance, est en cours dans le cas de la souille expérimentale en Baie de Seine, mais on n'a pas encore constaté de comblement significatif.

A diverses reprises, on a évoqué le fait qu'une tendance à la stagnation peut se développer dans les excavations et que des conditions anoxiques et toxiques pourraient s'installer. De telles conditions sont très peu probables en Baie de Seine où les courants sont importants ; on a observé dans le fond de la souille expérimentale le développement de ripple-marks qui montrent que même sa partie la plus profonde est parcourue par des courants significatifs empêchant toute stagnation. De tels phénomènes de stagnation sont probablement très exceptionnels et ne se développent que dans des mers ou des baies fermées, dépouvrues de courants de marée. Ils se produisent naturellement en Baltique ou dans les étangs du Languedoc Méditerranéen, mais ne sont pas signalés sur nos côtes, même dans des milieux relativement fermés comme la Baie de l'Aiguillon ou le Bassin d'Arcachon. Dans de tels cas, le flot et le jusant s'accompagnent de forts courants qui empêchent toute stagnation de l'eau. Il faut signaler enfin que les lieux où de tels phénomènes sont susceptibles de de produire sont des zones calmes où la sédimentation de matériaux fins prédomine et où par conséquent les gisements de sables et graviers de qualité sont exceptionnels ou absents.

En ce qui concerne les risques d'érosion de la côte liées aux extractions situées près du rivage, on apportera une attention particulière à l'examen de la bathymétrie de la région considérée et à l'étude de la houle et de la marée dans le secteur. Rappelons, en particulier, que des modifications de la bathymétrie entraînent des changements dans la réfraction, le déferlement et l'obliquité de la houle par rapport au rivage qui peuvent porter atteinte à l'équilibre du littoral.

On convient habituellement d'appeler matériaux fins ou fraction "fines" les matériaux dont la dimension est inférieure à 40 microns. Du fait de leur taille, ils se maintiennent longtemps en suspension, même en eau calme. D'autre part, même lorsqu'il ne s'agit pas de vrais colloïdes, ils se présentent, comme les argiles, certaines propriétés (floculations, adsorption). Dans les sédiments marins du plateau continental, ils sont essentiellement représentés par du quartz, de l'argile et de la matière organique.

D'une façon générale, l'argile et la matière organique étant des produits gênants, les exploitants chercheront toujours à extraire les dépôts de sables et graviers qui en contiennent le moins possible. Mais ceux-ci étant généralement présents, une certaine quantité sera, dans la plupart des cas, mise en suspension lors des opérations de dragage.
Il est donc indispensable avant de permettre l'exploitation, de demander au pétitionnaire de procéder à une évaluation de la teneur en matériaux fins de la formation exploité proprement dite ainsi que du substratum et de la découverte. Cette évaluation devra être faite sur un échantillonnage représentatif qui tiendra compte de l'hétérogénéité de ce genre de formation (coupes complètes du gisement).

En règle générale, néanmoins, la remise en suspension d'une quantité totale de matériaux fins est inévitale. Les tolérances qu'on peut accepter sont variables d'une région à l'autre : face aux grands fleuves dont la chargé alluviale est considérable, les tonnages remis en suspension par la drague peuvent se révéler négligeables ou marginaux. Par contre, à proximité des zones d'herbiers sur la Côte d'Azur ou sur certains points du littoral breton, où la transparence de l'eau favorise les déplacements algiques, des mises en suspension même faibles peuvent se révéler dommageables.

Dans les zones où des remises en suspension importantes pourraient être tolérées, d'autres facteurs tels que la vitesse et l'orientation des courants de fond et de surface doivent être pris en considération ; il faut connaître la vitesse de dispersion des matériaux fins, leur aire de distribution, les possibilités de resédimentation. Les expériences faites en Baie de Seine avec des tracers radioactifs montrent que la moitié des particules se sédimentent à moins de 2 km du point d'extraction. La loi de répartition étant exponentielle, environ dix pour cent sont encore en suspension à des distances de l'ordre de cinq à sept kilomètres. Ceci montre l'importance que revêt la détermination du pourcentage de fines de la formation draguée ainsi que la cadence et le volume des tonnages extraits. Certaines régions pourront être exploitées mais à des cadences telles que le niveau de trouble ne dépasse pas un certain seuil. Cette relation, cadence, teneur en suspension, est en cours d'étude en Baie de Seine.

III. Méthodes d'étude des souilles de dragage

La technique la plus élémentaire servant à connaître la morphologie du fond marin est l'utilisation du sondeur bathymétrique. Une onde acoustique est émise depuis le navire et se réfléchit sur l'interface eau-fond, ceci en continu le long d'une route donnée.

Pour avoir une vision plus large de la forme et de la nature du bord de la mer on utilise le sonar latéral (Figure 2). Dans ce cas on utilise une onde acoustique oblique qui, à l'image du soleil couchant, donne une ombre de tous les obstacles rencontrés. La surface couverte par une émission de sonar latéral est ainsi beaucoup plus grande que la "ligne" recouverte en bathymétrie. Cependant la mesure précise de la distance du navire à un réflecteur donné est surtout la hauteur de ce réflecteur qui est difficile sans de nombreuses et délicates précautions. Le sonar latéral est un excellent outil de reconnaissance rapide de la forme et des irrégularités du fond marin mais non un instrument de bathymétrie précise.

Lorsque le taux de sédimentation devient notable la bathymétrie reste trop imprécise ; on utilise alors une technique permettant de déterminer la faible épaisseur des sédiments nouvellement déposés, il s'agit de la micrométrie ou réflexion haute définition. L'énergie déployée pendant ces mesures se transmet selon une fréquence élevée (jusqu'à 14 KHz) et permet une résolution de l'ordre de 20 cm.

Enfin la caractérisation des sédiments passe par une phase de prélèvement par benne ou par petit carottier. Des analyses sédimentologiques classiques sont réalisées parallèlement à un inventaire de la faune pouvant s'installer dans les nouveaux sédiments.

IV. PROBLÈMES DE STABILITÉ DU LITTORAL

LIES AUX EXTRACTIONS

Pour mieux cerner le problème de l'influence de souilles d'emprunts de sédiments sur l'équilibre du littoral, une première série d'études en modèle réduit a été entreprise au L.C.H.F. en examinant le comportement de fosses de dragages réalisées à des profondeurs croissantes entre les fonds de -6m et de -23 m.

Ces études ont été faites en choisissant des échelles de similitude qui permettent de représenter correctement les mouvements des sédiments sous l'action de la houle et en contrôlant à partir des mesures faites en nature que les déplacements des matériaux artificiels sur le modèle sont conformes aux déplacements des sables mesurés pour des conditions hydrodynamiques déterminées. Une échelle en plan au 1/200 et une échelle des hauteurs au 1/75 ont été adoptées.

Des recherches en canal vitré inclinable ont tout d'abord permis de définir l'amplitude critique de la houle susceptible d'entraîner un début de comportement de...
la souille en fonction de sa profondeur. Les conditions de colmatage ont ensuite été analysées ainsi que les différentes formes prises par les dépots.

Cette étude en canal vitré, à deux dimensions, a été complétée par des recherches plus poussées en cuve à houle à trois dimensions permettant de prendre en compte l'ensemble des phénomènes hydrodynamiques qui interviennent dans les mouvements sédimentaires.

Dans tous les cas les houles reproduites se rapprochaient des houles naturelles, leur succession dans le temps s'effectuant sous forme de trains d'ondes dont la loi de répartition des amplitudes était conforme à celle des probabilités relevées en nature.

La mèche schématisée peut représenter une loi sinusoidale.

Les fonds étaient supposés en pente douce de 1,5% en nature, et représentés en cuve à houle entre les cotes +7,50m et -40m environ.

Les sédiments naturels ont été schématisés par des matières plastiques de densité 1,38 avec une répartition granulométrique conforme à celle mesurée en nature. Le diamètre moyen des grains artificiels était de 255 microns pour les études en canal, représentant des sables compris entre 60 et 300 microns en nature, et de 375 microns pour les études en cuve à houle représentant un sable de 250 microns de diamètre moyen.

Les souilles draguées avaient, pour les études en canal, un profil trapézoïdal, la largeur à la base étant voisine de 80 m et celle au sommet de 140 m, la profondeur étant de 5 m. Une telle souille représente une section de 550 m³/m. La souille était réalisée sur toute la longueur du canal.

En cuve à houle, un profil largement différent a été adopté afin de se rapprocher des indications fournies par les dragueurs. La souille expérimentale avait en nature une longueur de 800 m (4m sur le modèle), une largeur au sommet de 200 m et une profondeur de 6 m. Les pentes des berges de la souille étaient à 5% et le volume correspondait à une extraction de 800 000 m³ de matériaux (1 000 m³ par mètre linéaire).

Précisons que l'échelle des temps sédimentologiques applicable à l'ensemble des études en canal et en cuve à houle étant de 1/1500, une minute sur le modèle représentant environ un jour en nature.

De même rappelons que les amplitudes indiquées correspondent au H 1/10 et que les amplitudes significatives H 1/3 sont 1,2 fois plus faibles et les amplitudes moyennes 1,95 fois plus petites.

**V INTERPRÉTATION DES RÉSULTATS**

L'étude de quatre souilles implantées par des profondeurs de -6, -11, -16 et -21 et soumises à des houles du type "Golfe de Gascogne" permet de formuler les réflexions suivantes :

- Il existe pour chaque profondeur "df" de souille une amplitude critique Hc de la houle au-delà de laquelle la souille peut être soumise à un débitement. Cette amplitude critique (H 1/10 critique) est donnée par l'expression simplifiée :

  \[ Hc = 0,25 \text{ à } 0,30 \text{ df} \]

Autrement dit, une souille implantée par fonds de -6m sera stable, à l'échelon de 1 année, si les amplitudes H 1/10 ne dépassent pas 6 m et la période 12 à 14 secondes.

- Les volumes déposés dans les fosses augmentent très rapidement lorsque la profondeur de la fosse diminue et lorsque l'amplitude augmente. Dans le cas de houle frontale agissant au large du déferlement, le volume de comblement journalier peut être donné par l'expression très approchée :

  \[ V \text{ m}^3/\text{m/jour} = \frac{K}{(H + 10 - Hc)} \]  

\[ K = 3 \text{ dans les conditions expérimentales.} \]

- Sur une année complète les dépots par mètre linéaire de fosse seraient donnés, dans le cas du Golfe de Gascogne (bilan d'énergie annuel de 2,110^9 m² s2) et pour des profondeurs de la souille comprises entre -6 et -35 par l'expression :

  \[ V \text{ m}^3/\text{m/an} = 10^4 \text{ df} - 1,5 \]

Les dépôts annuels seraient de 315 m³/an par fonds de -10 m, de 80 m³/an par fonds de -25 m et seulement de 48 m³/m par fonds de -35 m.

- Les érosions côté terre de la souille se propagent jusqu'aux fonds de -5 m et atteignent des valeurs de 500 m³ par mètre linéaire par les fonds de -11 et -16 mètres. Pour la souille à -21 m ces érosions se trouvent limitées au voisinage immédiat de la fosse et ne dépassent pas 80 m³/m en deux cycles annuels.

L'ensemble de ces résultats recoupe les valeurs déduites des études théoriques du L.C.L.F., des mesures en nature à l'aide de traceurs radioactifs et des essais en canal vitré.

Pour une souille de caractéristiques
identiques à celles étudiées en modèle (800 m de longueur, 200 m de largeur; 6 m de profondeur) réalisées dans des fonds sablo-ux en pente douce de 1,5% et dont la granulométrie moyenne est de 250 microns, on peut prêsumer qu'il n'y aura pas de répercussions sensibles sur le littoral si cette souille est implantée par des profondeurs supérieures à - 21 m sous le niveau des plus basses mers et si les amplitudes H/10 des houles restent inférieures à 7 mètres.

Pour généraliser ces résultats et pouvoir donner les autorisations nécessaires aux extractions d'agrégats en mer il serait nécessaire de compléter ces premières recherches effectuées pour le compte du C.N.E.X.O. et de la Direction Départementale des Pyrénées Atlantiques, en examinant le comportement de souilles de dragages sous une action très longue (5 à 10 ans) de houles représentant exactement les variations de directions, périodes et amplitudes relevées en nature avec leur succession dans le temps.

De même il serait souhaitable pour étendre la validité de ces recherches à différents sites maritimes de représen-ter l'influence éventuelle de courants généraux ou de marées lorsque ces dernières existent et viennent se superposer aux actions de la houle.

Enfin, la forme des souilles de dragages et leurs conditions d'excision sont des paramètres dont il serait sou-haltable d'examiner les effets à long terme sur la tenue des lits tertiaux ainsi que l'influence de la nature des matériaux et de la morphologie des fonds.

Les premiers résultats obtenus montrent les espoirs que l'on peut avoir pour extraire des agrégats en mer et les premières contraintes que l'on doit s'imposer. Un outil de travail très puissant est disponible avec les installations du modèle réduit existant et il reste à la disposition de tous pour mieux aborder les problèmes d'extraction d'agrégats et éviter de perturber l'équilibre extrême-
ment fragile de nos littoraux.

VI CONCLUSIONS

Le but de ces études, est de permet-tre, dans un contexte régional, de choisir la meilleure implantation pour une souille d'extraction en fonction des différents paramètres :

Nature des fonds, régime des houles et des courants, profondeur d'eau, distances à la côte et nature de la côte.

Il n'y a jamais de cas général mais toujours des "cas particuliers", que l'on peut en France, individualiser en grandes provinces avec des points de convergences, Régime de la Manche, Régime des Côtes Bretonnes, Domaine du Golfe de Gascogne etc... Ainsi à chaque fois, une étude attentive des conditions du site sera nécessaire pour intégrer les données existantes.

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Notes:
- TW: Technique de l'eau.
Fig. 2 -- Principe du sonar latéral. Dans l'exemple ci-dessus, le sonar n'explore qu'un seul côté du profil suivi par le navire porteur.
COASTAL EROSION IN KERALA, INDIA—A CRITICAL APPRAISAL

EROSION CÔTIERE À KERALA, INDE—UNE EVALUATION CRITIQUE

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ABSTRACT

The Kerala coast of 560 km length trending in NNW-SSE to NW-SE direction is roughly straight with a few minor offsets in ENE-WSW direction. Out of this total length, nearly 360 km of coast in different reaches is under active erosion. The remaining reaches are either stable or under accretion. The Kerala coastal region is occupied by Archaean, Dharwars, Tertiaries and Quaternary formations. Geomorphologically the coast can be broadly classified into (i) Rocky coast, (ii) Rocky promontories with intervening sandy beaches and (iii) sandy coast.

This paper reviews the studies in a 230 km long reach of the coastal belt from Neendakara to Tanur. The coastal landforms are lagoon, estuary, sandy flat, mudflat, mixed flat, creek deposits, swampy and marshy zones, beach ridges, beach terraces, cheniers, spits and bars. The fluvial landforms include terraces, flood plain deposits, abandoned channels and channel bars. The studies indicate a loss of 100 to 400 m wide belt due to erosion in some reaches, gain of 50 to 1,000 m wide belt in certain reaches due to accretion and stability of coast in the remaining reaches during the last seventy years. Shore line variation in the last 120 years however shows a loss of 600 m wide belt in certain reaches.

The coast belongs to "compound shore" of Johnson and is considered unstable. Shortage of sediments appears to be the main factor of erosion. The other factors are steep slope of near shore, the wave angle and neotectonic movements. The mudbanks and rocky promontories in certain reaches contribute to erosion in the down drift side. The
seawalls do not appear to be quite effective. Artificial feeding is considered to be not feasible. Hence the need for multidisciplinary studies is stressed to understand the problem fully and to evaluate effective measures of control.

ABSTRAIT

La côte de Kerala d'une longueur de 560 km se dirigeant vers la direction Nord-nord-ouest Sud-Sud-est à Nord-ouest-Sud-est. Entre cette longueur totale, à peu près 360 km de côte aux atteintes différentes est sous l'érosion active. Les atteintes restantes sont soit stable ou sous accroissement. La région côtière de Kerala est occupée par des Archeans des Dharwars, des Tertiaires et des formations Quaternaires. Géomorphologiquement la côte peut-être largement classifiée comme (i) La côte rocheuse, (ii) des promontoires rocheuses avec des plages sableuses intervenant et (iii) côte sableuse.

Cette article revue les études dans une atteintes de 230 km long de la bande côtière de Neendakara à Tanur. Des formes de terre côtières sont: lagune, estuaire, plat sableux, plat à boue, plat mélange, dépôts de crique, des zones marécageuses et humides, des arêtes de plage, des terraces de plage, des cheniers, des crachats et des barres. Les formes de terre fluviales comportant des terraces, des dépôt des plains d'inondation, des canaux abandonnés et des barres à canaux. Les études indiquent une perte de 100 à 400 m de bande en largeur à cause d'érosion en certaines atteintes, un gain de 50 à 1000 m de bande en largeur en certaines atteintes à cause d'accroissement et stabilité de côte aux atteintes restantes pendant les dernières 70 années. Une variation de ligne de rivage dans les dernières 120 années cependant montre une perte de 600 m de bande en largeur aux certaines atteintes.

La côte appartient à 'rivage composé' de Johnson et est considérée instable. Mâque de sédiments apparaît d'être le facteur principal d'érosion. Les autres facteurs sont pente escarpée de rivage voisin, angle d'onde et des mouvements néotectoniques. Les bancs de sable et des promontoires dans quelques atteintes contribuent à l'érosion à côte diluvium d'aval. Les digues n'apparaissent pas d'être assez effectives. Nourriture artificielle n'est pas considérée d'être praticable. Ainsi la nécessité pour des études multidisciplinaires est accentuée pour comprendre le problème complètement et pour évaluer des mesures de surveillance effectives.
INTRODUCTION

Changes in the coast line are common geological phenomena all over the world. Depending on the dynamics of wave action, the disposition of the coast line to the dominant wind and wave propagation, the geomorphological set up of the hinterland, the morphology of sea bed, the eustatic or tectonic phenomena and such other related factors, there occurs either erosion or accretion; sometimes alarmingly large enough and sometimes imperceptibly low. India has a 5,700 km long coast line and erosion is reported at a number of reaches. But in the 560 km long coast line of Kerala stretching from Poovar (8°02'77°05') in south to Talapadi (12°46'74°58') in the north with a very high density of population, the loss of land due to erosion assumes great significance and becomes critical.

Out of the total 560 km coast of Kerala, about 360 km in different reaches is subject to active erosion (Fig.No.1). The remaining reaches are either stable or under accretion. The annual rate of loss of land in the reaches of erosion has been worked out to be ranging from 3 m to 5 m. The shore line variation maps prepared for some of the reaches indicate that about 600 m wide belt of beach has been lost due to erosion in the last 100 years. Realising the threat to this important coastal belt of Kerala, the Government of India constituted the Beach Erosion Board in 1966. As part of the recommendations of the Board, geomorphological and related geotechnical studies were taken up. This paper reviews critically the geomorphological and other aspects related to the coastal erosion. The need for further multidisciplinary studies to work out an integrated approach is briefly outlined.

Regional Geology:

The Kerala State consists of four physiographic units and these four from west to east are as follows:
i) The coastal strip of recent emergence consisting of estuaries, lagoons, barrier beaches, older spits and bars, former barriers, mudflats, creek deposits and swamp and marshy zones.

ii) Lateritic upland formed by low lying flat topped hillocks of 50 to 150 m elevation above mean sea level covering the Tertiary sediments and the Pre-Cambrian gneisses between the foot hills and the coast.

iii) The foot hill region of the Western Ghats comprising deeply dissected platform and including the stretch up to a height of about 1,000 m above mean sea level and

iv) The high ranges of the Western Ghats rising to an elevation of 1,000 to 2,500 m above the mean sea level.
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- Reaches under erosion
- Sandy coast
- Rocky coast
- Irregular coast

**Fig. 1.** Map of Kerala
Showing the Morphology Of the Coast
And Reaches Under Erosion

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The region is occupied by the formations of four major Super Groups namely (i) Archaean, (ii) The Dharwar, (iii) The Tertiary and (iv) The Quaternary. The Archaean crystallines comprising the charnockites, khondalites and garnetiferous biotite gneisses and migmatites constitute the high ranges as well as the foot hill regions. At places these formations protrude into the coast forming rocky promontories. Amongst the Archaean, the charnockites are considered to be the oldest. The Dharwarian schists trending in a WNW-ESE direction occur amidst the Archaean crystallines in the districts of Kozhikode and Cannanore. The granites forming the acid intrusives occur as domes and lenses in the gneisses and schists. The basic dykes trending in NWW-SSE, WNW-ESE and NE-SW directions traverse the crystallines and schists. The regional strike of foliation of the gneisses is NW-SE with local changes. The rocks are highly folded and isoclinal folding is a common feature. The Warakkallai and Quilon formations of Tertiary es comprise sandstone, limestone, clay, lignite and shale. These are almost flat bedded and have been subjected to high erosion. These extend as a narrow strip occupying the coastal plains of Trivandrum, Quilon, Alleppey, Kottayam and Cannanore districts.

The laterites are found to occur as cappings on the hills and ridges covering the Archaean crystallines, Dharwarian schists and Tertiary sediments between the foot hills region and the coast. The thickness of the weathered zone of which the laterite is the most common product varies from 2.0 m to as much as 50 m. The Quaternary sediments (other than residual soils) are confined to the coastal plains. The sub-Recent formations consisting of a good thickness of sands with shell fragments, black sticky clays and peat beds occur in the low lying areas fringing the Tertiary beds between Quilon, Kayamkulam, Kottayam, Ernakulam and Ponnani and also between Cannanore and Nileshwar. The sub-Recent marine and estuarine deposits are localised around the stretches of back waters and reclaimed low lands. The Recent formations include alluvial deposits and coastal sands. The alluvial formations deposited by the flood waters occur along the major river valleys and along the margins of the mudflats. The coastal sands include sandyflat and dune sands consisting of rows of sand bars alternating with marshy lands and lagoons which constitute the older sand bars and spits and recently formed parallel sand bars. At places there are small mounds or raised grounds consisting of fine grained reddish sandy loam known as 'Teris'.

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Morphology of the coast:
The Kerala coast trending in NNW-SSE direction is more or less straight with minor off-sets at places in ENE-WSW direction. The beach is very narrow to broad and varies in width from less than 10 m to 350 m. Black sands rich in illmenite and monazite constitute the beach between Neendakara and Aratipuzha (9°13’-76°25’). Quartz rich sands with or without pockets or admixtures of black sands constitute the beach material in the rest of the reaches of the coast.

Morphologically the coast can be broadly classified into (i) Rocky coast (ii) Rocky promontories with intervening sandy beaches and (iii) Sandy coast.

(i) Rocky Coast:
The rocky coast is predominant between Cannanore and Elathur with small stretches near Tangasseri, Warakallai and Kovalam. The total length of rocky coast is about 60 kms. The shoreline of rocky coast is highly irregular due to rocky headlands having differential resistance to erosion with intervening minor embayments formed at the sandy beaches (Fig.2). The rocky coast is formed mainly of gneisses and laterites and Tertiary sediment at places. Laterites and Tertiaries are in the form of small mounds and flat terraces. The occurrence of massive outcrops close to the sea has given rise to the cliffed coast. At a few stretches, the hinterland is plain but the rock exposures occurring near the shore make the coast rocky but not cliffy. Intertidal platforms are formed at the base of some of the cliffs. Stocks and needles are common in the near shore of these reaches of the coast.

(ii) Rocky promontories with intervening sandy beaches:
The shore line under this group is characterised by a combination of both irregularity and straightness. The irregularity is due to the rocky promontories and the straightness is due to the sandy coast intervening the promontories (Fig.3 and 4). This type of the coast has a cumulative length of about 50 km at the following locations.
(i) Between Tallechery and Kadalur point.
(ii) Between Elathur and Kollam point.
(iii) Between Kovalam and Chavara.

(iii) Sandy Coast:
The sandy coast stretching for over 80% of the Kerala coast is more or less straight trending in NNW-SSE to NW-SE direction (Fig.5). It extends between Elathur and Kovalam for a distance of 450 kms. interspersed with cliffed coast at places viz. Tangasseri and Warakallai and occupy a few stretches north of Cannanore and south of Chavara with narrow to moderately wide beach. The coast for a
major part of its length is formed either due to the present barrier or the older barrier. The total length of the sandy coast works out to 450 km.

The Seismic Refraction and Electrical Resistivity surveys in parts of Kerala indicate that the maximum thickness of sediments is 600 m between Ambalapuzha and west of Kayamkulam (Kartha et al, 1976). Of this 600 m only the top 100 m thick sediments are of Quaternary period (Flandrian-Holocene) and the underlying sediments to be of Aquatanian to early Bundigalian age of Tertiary Super Group.

Geomorphology of the Coast:

With the aid of air photos, the geomorphological studies of the 230 km long reach of the coast from Neendrakara in Quilon district to Tanur in Kozhikod district were carried out. The eastern limit of this study extends upto the crystalline boundary. Both marine and alluvial land forms are recognised in this area (Fig. 6 and 7).

Marine Land Forms:

The coastal land forms recognised are lagoon, estuary, sandyflat, mudflat, mixed flat, creek deposits, swampy and marshy zones, beach ridges, beach terraces, cheniers, sand dunes, spits and barriers. The older barriers and former spits and bars were inferred in this area by their geomorphic expressions (Fig. 6 and 7).

The sandyflat ranges in width from 0.4 to 8.0 km and occupy the area between lagoon and the lateritic upland or mudflat or mixed flat or creek deposit. There are beach ridges, beach terraces and narrow zones of mudflat or mixed flat within the sandyflat. These run generally parallel to the coast. The sandyflat (barrier flat) consists of dull white coloured sand with pockets of white quartz sands which are suitable for glass manufacture. The beach ridges within the sandy flat are 1 to 2 m high and 100 to 200 m wide. The beach terraces are 100 m to 700 m wide and occur as pockets within the sandy flat. The cheniers occur as linear bars of about 100 m width within the mudflat or lagoon formed at the mouth of the major rivers.

The mudflats consisting of black sticky clays and silty clays occur as pockets in the area between the sandyflat and the laterite upland. The mixed flats consisting of sand and clay occur as pockets in between the sandyflat and the mudflat. The creek deposits occur as pockets in the lateritic upland and in the transitional zone of the lateritic upland. The swampy and marshy zones occur as pockets and narrow zones within the mudflat zones or bordering the
Fig. 6  Geomorphological Map of the Coastal Belt Between 9° 59' 30" & 10° 06' 30" N

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Lagoons. The sand dunes occur in the backshore at places. A small hooked spit is under development at the mouth of the Periyar river. The barrier occurs in between the lagoon and the coast for a major part. The former spits and bars are inferred from the occurrence of linear bars within the mudflat or linear protrusions of sandyflat into the mudflat respectively.
uvial Forms:
The land forms formed due to uvial processes include river terraces, flood plain deposits, abandoned channels, oxbow lakes, and channel bars. Floods plain channels and terraces upto 5.5 m height occur on either side of the major rivers. The rivers have youthful course in the hill ranges and the foot hill regions, sinuous course in the upland region and meandering to old courses in the plains. Most of them...
debouch into the lagoons or estuaries. The major rivers like the Periyar, Pambayar, Achenkoll etc. have built up small or incipient deltas represented by thick pockets of flood plain deposits in the reach between the upland region and the marine deposits.

**Progradation and Retrogradation of the Coast:**

The geomorphological studies indicate that the shore line has undergone many changes during the last 60-70 years. A 100 to 400 m wide belt of land was lost (retrogression) due to erosion in certain reaches, 50-1000 m wide belt of land was gained (progradation) due to accretion in select reaches, and in certain reaches the coast has been stable during the last six to seven decades. The shore line variation maps of certain reaches indicate that about 600 m wide belt of land was lost during the last 100 to 120 years (Fig. 8 and 9).

The beach is generally very narrow in the reaches of erosion or in the reaches subjected to erosion earlier. In these reaches the shore line is not smooth and straight. It has embayments and intervening sandy heads. The beach is generally smooth and wide backed by broad backshore in the reaches where the coast is free from erosion or where there is accretion.

**Structures along the Coastal Belt:**

From the air photo studies, five sets of lineaments trending in NNW-SSE (parallel to the coast), ENE-WSW (perpendicular to the coast), NW-SE to WNW-ESE (parallel to the general trend of the gneisses of the hinterland), NE-SW, N-S and E-W were inferred in the coastal belt. The lineaments may relate to the Precambrian structures. The lineaments trending in NNW-SSE and ENE-WSW directions seem to have been affected by the neotectonic movements as evidenced by off-setting of the coast line, acute bends in the stream courses and lagoons and minor elevation differences on either side of the lineaments. These neotectonic movements appear to be still active. The erosion appears to be more severe in the reaches affected by ENE-WSW trending lineaments, while there is progradation of the coast to the west of NNW-SSE trending lineaments.

**Type of coast:**

The geomorphological studies show evidences for both submergence and emergence of the Coast. In the Sandy coast reach, the older barrier, former spits and bars indicate recent emergence of the coast. The lagoons, mudflats and the drowned valleys occupied by the creek deposits in the lateritic upland indicate submergence of the coast. In the reaches of rocky coast, the former spits and
Fig. 8. Shore line changes between Manassery and Kannamali
9°-52' To 9°-55'
D.V. Raju and K.C.C. Raju

Fig. 9. Shore line changes between Thottappally and Purakkod
9°-18'-36' To 9°-21'-30'
D.V. Raju and K.C.C. Raju
bars indicate emergence of the coast while drowned valleys, lagoons and mudflats indicate the submergence of the coast. The Kerala coast shows 'double shore line' feature with the inner shore line represented by the highly irregular and rocky coast along the eastern margin of the lagoons and the mudflats and the outer shore line represented by the present barriers or older barriers. The 'double shore line' feature of the coast with evidences for both emergence and submergence correspond to the 'compound' shore line of Johnson (1919). The coast might have been submerged during the quaternary period (Flandrian transgression). The river terraces and other geomorphic evidences show that the sea level during the transgression was five to six meters above the present sea level.

According to the genetic classifications of the Shepard (1963) the straight sandy coast reach belongs to the 'Secondary Coast' mainly developed by marine action while the rocky coast might be termed as 'primary coast'. The off-shore fault trending in the NNW-SSE direction might be a contributory factor for the development of this straight coast in both sandy and rocky coastal reaches. The neotectonic movements along this fault have probably rendered the coastal belt unstable. So from the straighness of the coast coupled with evidences both for submergence and emergence, it may be concluded that the Kerala coast might have developed due to the combination of both tectonic and eustatic activities which might be still operative.

Causes of Erosion:

The 40 and odd west flowing rivers of Kerala have not only short courses but also have youthful character for a major length of their courses. These rivers develop sinuosity as soon as they reach the lateritic upland and are forced to develop meandering in the plains followed by old stages. In spite of the sheet erosion of the laterites and soil which occupy major portion of the catchment areas, these rivers carry comparatively little sediments because of their short courses. These sediments get deposited in the lagoons and estuaries into which these rivers debouch before joining the sea as a result of which they commonly develop migratory character near their mouths. So there is not much supply of sediments by the rivers to the sea for reworking and redeposition. Due to this short supply of sediments to the sea by the rivers the unspent energy of the waves and currents is diverted towards the erosive action of the coast. This problem is likely to become active in future when all the hydel and irrigation projects
now under construction and contemplation are completed.
2. The steepness of near shore slope and critical angle at which the waves strike the coast are considered to be responsible for severe erosion in certain reaches.
3. The neotectonic movements in the coastal belt and in the near shore where they give rise to 'slips' or 'creep' of the sea bottom sediments may also be responsible for erosion in certain reaches of the coast.
4. The presence of mudbanks in the nearshore/innershore region, a peculiar feature of the Kerala coast may also contribute to erosion. They cause erosion on the down-drift side and accretion on the up-drift side.
5. In the reaches of rocky coast, the promontories or headlands also cause erosion on the down-drift side and while the stretch in the up-drift side is free from erosion or is under accretion.

Protective Measures:
The total length of the reaches for which protective works are carried out is of the order of about 250 km in length. The main protective measure has been in the form of seawall. Groyne are constructed at places. The seawalls designed without consideration to the direction and height of the waves, the angle of the slope of the beach and neotectonic movements seem to have been affected. In course of 4-6 years time, these are found to have sunk along the sloping section and collapsed near the base of the vertical section on the seaward-side giving way to the waves to pass or spill over the sea wall. As a result of the sinking and collapse, these seawalls appear to have lost their effectiveness in controlling the erosion. Likewise the groynes constructed without much consideration to the longshore drift and current movement also appear to be of not much effective in controlling the erosion. Though the main factor for the erosion is shortage in the supply of sediments to the sea for reworking and redeposition on the beach, the artificial feeding of sand is considered to be unfeasible in view of the large quantities required and the cost involved to protect the long coast line. Of late the use of sand filled coir bags with rubber coating inside as an armour wall on the existing rubble seawall is found to be successful to control erosion in small stretches.
The offshore artificial bars adopted off Japan and American coasts are being tried on experimental basis. The knowledge of neotectonic movements, current movement, wave pattern and nature of sediments are necessary to
plan and design these offshore artificial bars. Any single type of protective structures may not be able to protect the coast from erosion. A combination of groynes with seawalls in between the groynes and offshore artificial bars in the reaches of severe erosion may be able to protect the coast from retrogression.

At present, the saline waters advance into the coastal belt affecting both surface and subsurface hydrological regime in the area. In order to evaluate the influence of the protective structures on the hydrological regimes of the area, it is considered necessary to study the saline-water/fresh water interface relationship.

Need for Multidisciplinary Studies:

The net effect of erosion/accretion is controlled by either individually or collectively by:

i. waves, tides and ocean currents.

ii. eustatic oscillations.

iii. movement or transportation and deposition of sediments in the foreshore and nearshore.

iv. neotectonic movements in the intracoastal zone and

v. geological and geomorphological characteristics of the coast and hinterland.

Studies on all these aspects are considered essential for proper evaluation of the factors finally resulting in erosion/accretion. Therefore, coastal erosion studies involve the collection, interpretation and correlation of data of various disciplines of science. So a multidisciplinary study covering geology, geomorphology, geophysics, hydrology, oceanography, coastal engineering etc. are required to assess the causes and evolve measures to control erosion. Instead of diversifying the attention to the entire coast, the multidisciplinary studies are suggested to be carried out in selected reaches of erosion, accretion/stability and of mudbank area, while preliminary studies can be carried out along the entire coast.

The type of investigations suggested in the selected reaches and for the regional surveys may be classified into onshore and offshore studies. The onshore studies include the collection of data on erosion, preparation of geological, geomorphological and structural maps; preparation of shore line variation maps; geophysical and hydrological surveys; study of eustatic oscillations and seismicity and neotectonic movements. The offshore studies include bathymetric studies, collection of data on tides, waves and currents, study of movement of sediment in the near shore/fore shore, geophysical surveys, drifting and study of mudbanks.
Summary and Conclusions:

Out of the 560 km long coast of Kerala, different reaches cumulatively constituting 360 km in length are under active erosion. The remaining reaches are either stable or under accretion. The geomorphological studies carried out in a 230 km long reach indicate loss of 100 m to 400 m wide belt of land, due to erosion in certain reaches, gain of 50 to 1,000 m wide belt of land in certain reaches due to accretion and stability of coast in the other reaches during the last seventy years. Shore line variation maps show loss of 600 m wide belt of land during the last 120 years in certain reaches.

The Kerala coast belonging to 'compound shore line' of Johnson might have been developed due to the combination of both tectonic and eustatic activities. The coast is still being subjected to these activities and is unstable.

Shortage in the supply of sediments to the sea for reworking and redeposition by it appears to be the major factor for erosion. The steep nearshore slope, the angle at which the waves strike the coast and neotectonic movements are other factors responsible for erosion in certain specific reaches. The presence of mudbanks and rocky promontories cause erosion on the down-drift side.

The protective structures of sea wall and groynes constructed for about 250 km length of the coast do not appear to be effective in controlling erosion. Artificial feeding of sand is not feasible. The studies indicate that the erosion is complex and is the result of collective action of waves, tides and currents, transportation and deposition of sediments in the foreshore and nearshore, eustatic oscillations, neotectonic movements and geological and geomorphological characteristics of the coast and hinterland. Therefore a multidisciplinary studies covering geology, geomorphology, geophysics, hydrology, oceanography and coastal engineering etc are needed to identify the causes and evolve measures to arrest erosion.
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SHORE PROTECTION IN ALMAZORA COASTS (CASTELLON, SPAIN)

PROTECTION POUR LA CÔTE D’ALMAZORA (CASTELLON, ESPAGNE)

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ABSTRACT

A new protection system is proposed for a very regressive shore based on the knowledge of its coastal genesis and geomorphology, the littoral processes and the incidence of the Castellón harbour.

ABSTRACT

La nouvelle défense pour cette érosion côtière a défini après l'étude de la géomorphologie, des processus littoraux et de l'impact du port de Castellón.

I. INTRODUCTION

Spanish Mediterranean coasts are becoming an attractive focus for several centuries now, since the different environmental circumstances began to favour a general trend of populations towards coastal areas (6). This general trend has been accelerated along this century because of the economic development and the spreading of spare time (6,16). The different factors fostering coastal and littoral occupation, however, have affected, selectively the different places of these coasts, modifying somehow the trends along the time (9). Some parts of the littoral are reaching their best for touristic development now while others have reached it years ago though for agriculture and other aims; as the stretch of coast here studied, where the agricultural vocation and riches have probably contributed to a minor attention to their beaches. That reason, the presence and development of the harbour of Castellón, and the spreading of the industrial zone around it, may explain the lack of touristic interest and, as a consequence, the relative carelessness in which this shore has remained for too many years, suffering one of the most spectacular erosion processes in all our Mediterranean coasts. The efforts trying to pa-
lliate it have no obtained good results.

The present knowledge of influence of the littoral dynamics on littoral processes (4,5,16) permit to study them acceptably well in a particular shore stretch, and to establish the permanent or cyclic character of the shore changes, and thus, their reversibility degree. The case here studied is somehow a simple one, as it will be shown, since the littoral dynamic produces a clear direction for the net longshore transport (3) in it, being directly meant for the concepts of the Simple Coastal Forms Genetic Classification (15), which are more useful than other wider ones (11). In the comprehension of the littoral processes, previous advance in wind knowledge (7) and in their application to wind wave forecasting (14,16) have been taken into account.

II. DESCRIPTION AND GEOMORPHOLOGY

The Northern alignment of the "Iberian" chains ends towards the Mediterranean sea in the "Maestrazgo" wide formation, which is principally constituted by thick lime and loam Cretaceous strata. It approaches notoriously the coast imposing its orientation and separating the Valencian Oval from the Ebro Valley up to the proper shoreline. Just in the Northern part of the Oval is the area studied which, as the rest of it, is principally constituted by quaternary sediments from different origins forming the wide "Plana" of Castellón. At present the most important river running down on those deposits is the Mijares which, like the others, has probably had a much more important activity during the past Quaternary. Their function as feeding rivers of this coastal area is not accurately defined because of the lack of Quaternary climatologic data, but the importance of the modelling capability of the Mijares river is obvious considering its relatively long course from the "sierras" of Teruel. It has carried important run-off and abundant deposits towards the "Plana" forming a notorious delta. Though less important, the role of the other rivers is not negligible and if some or all of them ever joined the Mijares river to form one only river course it is not easy to state now, when the different delta fronts can be traced.

In any case, the advance of this coast from the retired positions at the foot of the sierras, up to the present arrangement, has been probably caused by the alternate combination of deltaic and littoral barrier formation processes. The delta fronts themselves may have behaved as littoral barrier supports to close littoral lagoons. This kind of processes has recurred several times due to an important torrential activity, fed from the interior massifs, and it probably began at the end of the Tertiary, during the Pliocene to Pleistocene transit, when a special weather occurred. An important posterior activity, during the Pleistocene and in relationship with the different glacial periods, has led to the present arrangement of this coast, in which it is possible to define a relatively individualized stretch between the Orpesa cape - one of the Iberian chain foothills reaching the shoreline- and the Mijares deltaic front, though none of both ends can be
considered as a total transversal barrier (15). Actually all the Valencian Gulf coast forms a unique natural physiographical unit, up to the San Antonio cape, and only the old Cullera Isle (now a small cape) constitutes a relatively notorious geometric singularity. However, several ports in this long natural unit have become more important singularities than that one, cutting the littoral longshore transport, and even behaving, as in the case of Valencia port, as a total barrier to it.

The port ("Grao") of Castellón has been established for centuries in the middle of the defined stretch but it has been a rather little port -principally for fishing- up to the second half of the last century in which it has begun to develop; so, in this century it has undergone two different enlargements, in 1906-1909 and 1926-1928, becoming an obstacle "geometric singularity" (15)- more important than both the cape and the delta. This artificial transversal barrier has caused several transformations which will be analyzed in the following chapter.

In accordance with the described genesis of this coast all this area shows a rather flattened bottom and offers several continental and littoral formations from the foothills up to the shore, though all the successive lagoons and barriers but the last are filled and covered by torrential sediments, clays with some dispersed pebbles proceeding from older layers. This continental sedimentation is completed with some little fluvial terraces which only in the Mijares river reach an appreciable extension. At the South of the Mi-
It has led to several local shore protection actions which have had a variable effectiveness, but most of them have become minor singularities of local negative impacts and/or behaved as transmitters of the erosive process, as a consequence of the structural difficulties to plan and manage a global shore protection system. The stretch studied has been implemented with several successive elements for shore protections without satisfactory results up to now; they have stopped the shoreline regression for some time but it has been at the expense of the bottom erosion which increases the wave action against the shore, as it will be shown later. At present the regressive character of this stretch is shown in the damages of the revetment and groins and in its decreasing double-supported, landscapelly degraded, bermless beaches, in which the proportion of sands has diminished, in favour of gravels and pebbles. At the same time the windward beach supported on the principal breakwater of the port goes on growing up at the expense of the southward littoral drift.

The principal sources of materials in this area were the aforementioned rivers and riberas, particularly the Mijares, and the cliffs of Oropesa cape, but it was probably more important than these the littoral southward drift, whose materials proceed from the Ebro delta and river. The present form and size of the port of Castellón avoids all this littoral drift reaching the beach, foreshore and onshore, before the Mijares mouth at least; therefore the only active source of materials for the stretch between the port and the delta are the river and the products of the erosion of the very delta and shore. The role of the Ebro materials in the construction of these coasts, up to San Antonio cape, has been mentioned many times (1,9), basing on the obvious dominance of the Southwards littoral transport all along the Valencian Gulf coast up to some point in which the shoreline orientation changes the direction of the wave currents.

III. LITTORAL DYNAMIC

The lack of registered or observed data of the sea for these coasts is mentioned in other paper of this Congress (8). The general atmospheric circulation on Eastern Spanish Coasts and the way in which that difficulty has been coped with for Santa Pola bay is also described in it. In the same way, the Routing Monthly Charts (2) have been taken into account in this work. Although none of the observational areas of these charts may be considered suitable for ALMAZORA coasts, it has been proposed an interpolation between the two closest areas—one onto the sea of Alger at the South, and other onto the Lions Gulf, at the North—based on the average of their respective data (7). Though breezes and other local winds can affect littoral processes both directly acting over dunes and berms and through generating currents and waves, it has been shown, on the one hand (7), their scarce influence on the littoral transport in this area and, on the other hand (see chapter II), that the present situation of this stretch of the shore makes the direct aeolian processes irrelevant.
Therefore the annual directional wind wave regime (see Appendix) has been obtained starting from the aforementioned Admiralty interpolated wind data, which may be considered a good approximation for sea and swell winds. The Integrated Method (14), modified (7) in relationship with the sectorial definition of the Fetch has been followed in such an orientation (fig. 6).

It is also possible to define thus the seasonal wind and wave regimes. During the autumn (only October, 7), winds and waves are weak and from all directions, but S components seem to be lightly dominant; it is the unique season in which northward longshore transport prevails. In winter and springtime (November-May) the dominance of NE and E components is very obvious and winds and waves grow up in frequency and intensity; during this period the littoral transport acquires a great importance and the erosive processes are very notorious. All along the summer (June-September) winds and waves go down but NE and E components go on prevailing.

An average annual resultant wave is obtained and it forms an important angle of incidence with the perpendicular to the general orientation of the shoreline. In figure 1 the direction of this resultant is shown comparing it with the supposed actual resultant, obtained as a perpendicular to the Valencian harbour supported beach, which may be considered to be next to the balance position (R.M. versus R.M.y).

IV. COASTAL MODIFICATIONS

The urgency of this study was imposed by the great damages produced on the revetment and it avoided the implementation of an adequate observation for over one year to notice the seasonal changes. Therefore it was necessary to be restricted to the available documentation which was obtained from three sources:

a) The aerial photographs of five different flights (1947, February; 1956, June; 1965, August; 1972, June; and 1977, March), such as they were restored in (3) but taking into account their seasonal differences (7).

b) Several plan designs of the port of Castellón in which the bathymetry around it is shown (1876, 1882, 1898, 1906, 1909, 1966), and other available data from both the coastal headquarters and the harbour office.

c) Two different complete bathymetries: one (1981, February) was made for this study; the former is from a 1977 nautical chart, by Spanish Navy Hydrographic Institute.

The two former sources permitted to know that the revetment was built for the first time between 1947 and 1950, the groin no 1 (fig. 2) within the 1950-58 period, the groins no5 5, 10, 13 within the 1965-1972 period, and the rest of them between 1972 and 1977. Besides, the first source may be used to estimate the rate of deposits to windward of the port, and the second source shows that the first enlargement of the port was built in 1906-1909, and the second and definitive one, in several stages, after 1926 when the northern breakwater was built.

As the average annual advance of the shoreline to windward of the port...
is increasing from 1947 to 1977 up to reach a total one of up to 100 meters (3), in the stretch studied the shoreline average regression in the same period was about 30 meters, though the process seems to have stopped before 1972 (it has been advised and it will be shown that such an appearance proved to be wrong. Actually, the strong quarystone revetment is suffering important damages during the last years, as well as the promenade and the "villas" and other properties). Around the port it has been possible to determine the shoreline variations since 1876 in six different transversal profiles from the plan designs and they are shown in TABLE 1, in meters from a reference line. Profile P-2 shows the accretion close to the Northern breakwater; it was continuous since 1882 but between 1876 and 1882 some regression appears. Profile P-3 shows the erosion to leeward and close to the old southern breakwater up to the first enlargement of the port (1906) when it remains in the area of the little leeward beach; since then some accretion has occurred up to the second enlargement when a new southern breakwater was built southward from the previous one. The influence of the port evolution on the shoreline position is as obvious at a short as at a long distance.

A valuation of the rate of deposits at the North of the port supposing constant slopes (3) gives about 130,000 m³/year between 1947 and 1977. The estimate of the littoral drift using the wave data of the World Meteorological Center for Valencia and the C.E.R.C. formula (3), and comparing it with the accretion
TABLE 1

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<th>Year</th>
<th>1876</th>
<th>1882</th>
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<td>210,00</td>
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</tr>
<tr>
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<td>7,50</td>
<td>120,00</td>
<td>180,00</td>
<td>240,00</td>
<td>329,00</td>
<td>587,50</td>
</tr>
<tr>
<td>P-3  (+220 m.)</td>
<td>60,00</td>
<td>-15,00</td>
<td>35,00</td>
<td>59,00</td>
<td>**</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P-4  (+320 m.)</td>
<td>-40,00</td>
<td>-20,00</td>
<td>(**)</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>P-5  (+400 m.)</td>
<td>-40,00</td>
<td>-20,00</td>
<td>(**)</td>
<td></td>
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</table>

D stands for the distance in meters to a reference point, in the interior of the harbour. Sign minus (-) means northward and sign plus (+) southward from it.

** These profiles remain interior to the harbour after the last enlargement.

volume in Sagunto and in Castellón harbours, permits to establish a rate of 525,000 to 575,000 m³/year. On the other hand, the bathymetries of the port plan designs have permitted to establish the transversal profiles since 1876 and, from them, the changes in the volume of sands around the port. From these changes an average rate of 29,6 m³/m.year has been obtained for the period of 1909-1966, as the correspondent value from (3) is 27,8 m³/m.year for the period of 1947-1977; other average rates obtained for different periods have been 33,3 m³/m.year in 1876-1906, and 35,3 m³/m.year in 1906-1909; the concordance of these results seems to be rather acceptable.

The extrapolation of the previous results, around the port, to all the beach northwards (about 4,500 m. length), for different hypothesis about the variations of the slopes and the shorelines position permit to confirm an average rate of deposits by lineal meter of about 30 m³/year, during the last century, corresponding to nearly 600,000 m³/year for the littoral drift.

Finally, comparing the profiles which have been determined in 1981, Februry, with those obtained from the aforementioned navy chart (figs. 3a & 3b), the erosion rate, taking into account different hypothesis about the width of the longshore transport zone, is estimated over 113 m³/m.year, without exceeding probably 130 m³/m.year. Supposing that this value corresponds to the total transport capability, which means that in the port all longshore transport is either deposited or carried out to unavailable depth, and that in the stretch studied the littoral drift is saturated again, that value fits rather well the net longshore transport rate of about 600,000 m³/year. Southward, previously obtained.

The application of the C.E.R.G. formule in accordance with the predicted wind wave regimes (see chapter III and Appendix) gives a net southward longshore transport ($Q_n$) of about 596,000 m³/year and a total (southward plus northward) longshore transport ($Q_n$) of about 1,127,000 m³/year — About 861,000 m³/year southward ($Q_{ns}$) and about 266,000 m³/year northward ($Q_{sn}$). The values of these magnitudes for Sagunto harbour, some kilometers southward in the same coastal area, are different and somehow signific
cant; especially the northward transport value. Those values, obtained (3) from the scarce data of the world Meteorological Center, are the following:

\[ Q_{NS} = 571,000 \, \text{m}^3/\text{year}; \quad Q_{SN} = 48,000 \, \text{m}^3/\text{year}; \quad Q_{P} = 619,000 \, \text{m}^3/\text{year}; \quad Q_{N} = 524,000 \, \text{m}^3/\text{year}. \]

But all these values have been obtained applying to the C.E.R.C. formula a 0.76 experimental coefficient, which was deduced for Valencia port area. In any case the results from the different sources are somehow approximate and more accuracy obviously wants a longer and wider research.

V. CONCLUSION AND RECOMMENDATIONS FOR SHORE PROTECTION

Though the urgency for adopting a solution avoided a complete observation throughout at least one annual cycle and reduced the possibility of obtaining more experimental data, the knowledge of the processes and of their more determinant factors which may be deduced from the elements developed in the previous chapters is good enough to propose some available action for the shore protection.

It has been shown that the shore stretch studied is placed in a typical plain coast, developed during the quaternary age through complex processes including deltaic and littoral activities in a dialectical relationship. Littoral barriers (spits and cordons) and lagoons have increased their relative importance during the epochs when the Ebro activity increased, as it is the case with several notorious lagoons appeared in historical age, as "el Cuadro" of Castellón and "la Albufera" of Valencia; in other moments the delta fronts prevailed. Lately both kinds of activities have decreased because of the works to a better water management, and as a consequence, the sources of materials have been reduced; therefore the general circumstances have limited strongly the possibilities of the traditional trend of the shoreline to advance into the sea. And, in some cases, the very shoreline has become a source through undergoing erosion of the wave processes action.

Recently, other human actions on the littoral areas have made the problems of the erosion more acute because of both their interference in the wind and wave littoral processes (it is the case with the port of Castellón) and the reduction they cause in the riverbed materials (in the Mijares principally). This erosion has been particularly intense in the stretch studied, and it has mainly been due to the impact caused by the port enlargement through this century, as shown in the previous chapter.

The present arrangement of the port makes these stretch profiles extremely underfed, as they are undergoing the action of a rather defined southward net littoral dynamic, with a light northward component. The littoral materials passing in front of the port from the north cannot reach this stretch at all, and the only available source is the very Mijares delta. Therefore the built revetment cannot be an efficient shore protection system; it has only delayed the shoreline regression but has not stopped the erosion. At present the erosion not only affects the shoreline and the beach but the breaker zone, even its seaward zone too.

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On the other hand the size and efficiency as a barrier of the port and the lack of a by-passing disposal make the transversal groins built since 1965 useless because of the impossibility of getting a sufficient feeder source by trapping the littoral drift. Their initial apparent capability for shore protection only becomes, in those conditions, a transitory delay of the general regression by changing the erosive processes; for a time the landward onshore erosion is substituted by a seaward onshore and offshore erosion up to reach the instability and inefficiency of the groins; since then onshore and shoreline erosion follows. To make matters worse the southern groin has a length that avoids the delta materials to feed the stretch when storm waves impose a northward littoral transport. Therefore, neither the rubble-mound revetments nor the groins have stopped the erosion process and in such a way that the present situation of the shore permits a harder wave attack: the more erosion process advances, the deeper and steeper is the onshore zone, the higher and nearer are the breaker waves and the more intense is the erosion. Perhaps both the present situation and the behaviour of the shore protection disposal would have been different if a sand by-passing system had been implemented through the port from the beginning, but without it, all the shore-protection actions built have proved their unsuitability.

So, though a sand by-passing system seems to be the first one to have been provided to avoid or stop the erosive process, that solution is surely insufficient at present, when this coastal stretch needs not only a nourishment but a shelter disposal which reduces the power of the littoral dynamic. Therefore the different solutions considered and the proposed one are based on offshore breakwaters and islands with an intermediate orientation between the perpendicular to the annual average resultant wave and the shoreline orientation. For all of them the slope and the nature of the bottom have been taken into account, and the groins already built are to be used for the construction proceeding, in order to reduce both the budget and the ecological impact of the building. Four different alignments have been considered, at 100-120 m. (about 3 m. depth), 150-200 m. (about 4 m. depth), 200-250 m. (about 5 m. depth) and over 300 m. (under 6 m. depth) from the shoreline respectively (figs.). And among the several solutions analysed, based on elements of these four alignments, the one shown in fig. 4 has been chosen; all of its offshore breakwaters are under 160 m. from the shoreline and on bottoms under 4 m. deep. The number of breakwaters is the lowest being compatible with their efficiency and with the other aforementioned criteria related to the slopes, the position and suitability of the existing groins and the direction of the average wave (Rm). The separation between breakwaters and their length shall avoid the unrefracted waves reaching the foreshore.

APPENDIX: LITTORAL DYNAMIC

In accordance with theory of the wave forecasting methods and, particularly, with the "Integrado" modified
The wind wave regimes for maritime directions of incidence has been defined for great depths, (fig. 5), from the well established wind regimes (discarding breezes, 7), taking into account the Fetch of figure 6. From those regimes an annual average resultant wave has been determined by applying the C.E.R.C. (16) longshore transport rate formula (in decimal units):

\[ Q(\text{m}^3/\text{year}) = 2045 \times 10^{-3} \cdot f \cdot \frac{K^5}{2} (\text{m}) F(\alpha) \]

To get \( f \) value, \( f = (1 - F_x) K_0 \cdot K_x \), where

\[ K = \text{no of observations in the sector} \], \[ n_0 = \text{total no of observations} = 1086 \]

and, \( K_x = \text{real amplitude of the sector} \text{ theorical ampl of the sector (45\degree)} \text{to consider that some of the octants are reduced by the geographical configuration.} \]

\( F_x \) has been obtained from fig. 5 where all regimes have been considered shortened by the \( (H_o)_{\text{max}} \) corresponding the highest theoretically observed wind speed.

Five incident directions have been considered and the \( \alpha_o \) value of each is the average one of the two extreme values of the sector. The first sector, corresponding to the North, is worthless because of the presence of the port of Castellón (figure 2). The sector (octant) corresponding E direction has been subdivided in two different sub-sectors, each with its \( \alpha_o \) value corresponding to a different longshore transport direction.

a) N direction: \( \alpha_o = \text{av.} \) (900, 800) = 850

\[ K_0 = 0,1070 \; ; \; K_x = 0,2222 \; ; \; (H_o)_{\text{max}} = 2,65 \text{ m.} \]

Wind Wave regime:

\[ H_o = 0,1669 - 0,8406 \ln(1 - F_x) \]

b) NE direction: \( \alpha_o = \text{av.} \) (800, 350) = 57,50

\[ K_0 = 0,1246 \; ; \; K_x = 1 \; ; \; (H_o)_{\text{max}} = 4,79 \text{ m.} \]
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<th>$Q_1$ (NE)</th>
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</table>

$Q_T = 1.127 \cdot 607 \text{ m}^3/\text{year}$

\[ H_0 = 0.2541 \cdot 1.0959 \ln(1 - F_x) \]

**Wind Wave regime:**

- $H = 0.2541 \cdot 1.0959 \ln(1 - F_x)$

**c) E direction:**

**Wind Wave regime:**

- $H = 0.2541 \cdot 1.0959 \ln(1 - F_x)$

**o) first subsector:**

- $\alpha = \text{av.}(350, 00) = 17.5^\circ$
- $K_0 = 0.1245; K_f = 0.7778; (H_o \max = 8.1 \text{ m})$

**c2) second subsector:**

- $\alpha = \text{av.}(00, -100) = -52^\circ$
- $K_0 = 0.1245; K_f = 0.2222; (H_o \max = 8.1 \text{ m})$

**d) SE direction:**

- $\alpha = \text{av.}(-100, -550) = -32.5^\circ$
- $K_0 = 0.0709; K_f = 1.1 (H_o \max = 4.15 \text{ m})$

**Wind Wave regime:**

- $H = 0.3462 - 0.9655 \ln(1 - F_x)$

**e) S direction:**

- $\alpha = \text{av.}(-550, -900) = -72.5^\circ$
- $K_0 = 0.0761; K_f = 0.7778; (H_o \max = 3.33 \text{ m})$

**Wind Wave regime:**

- $H_0 = -0.2614 - 0.7962 \ln(1 - F_x)$

The values of the Table 2 have been obtained with the above data. A vectorial composition may be obtained in which each component vector has the direction of its $\alpha_o$ value and the length of the $Q$ value determined in the Table. Though it is not very significant, the net longshore transport obtained in this way is about of 600,000 m$^3$/year and, independently of the accuracy of this formula for these coast, this vectorial composition may be accepted because its inaccuracy will affect all directions in the same proportion. The aim of this composition, looking for an annual average resultant wave R.N., is not to define an accurate longshore transport rate but to establish an incidence direction for annual average wave.
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GEOGRAPHICAL FETCH
SECTORIAL DEFINITION

Fig. 6

SHORE MANAGEMENT IN SANTA POLA BAY (ALICANTE, SPAIN)

LA CONDUITE DES BAIES A SANTA POLA (ALICANTE, ESPAGNE)

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ABSTRACT

A study of the possible implementations for shore protection and arrangement in order to improve its ambiental and leisure conditions is developed for Santa Pola bay. It is based on a detailed knowledge of the geomorphologic evolution and the factors contributing to the particular littoral dynamic and processes of this coastal zone.

ABSTRAIT

On a étudié les possibles actuations pour la protection et amélioration de la côte et les plages de la baie de Santa Pola. Pour cela on a étudié auparavant la geomorphology de la zone et son évolution aussi que les différents factors qui affectent la dynamique et aux procés littoraux dans cette côte.

1. INTRODUCTION

Santa Pola Bay Coasts are among the coasts which have undergone a most intensive recent overcrowding. Though not being coasts with widely generalized settlements since the beginning of the centripetal movement of populations in the Peninsula, they had had a long use, for fishing activities at the beginning, and based on salt exploitation afterwards, taking advance of the protection of the Santa Pola Cape and the Tabarca Isle. Being a typical plain, lagoon coast, it could not offer any good wide beaches up to now: the cape is not a very available place for permanent beaches of a certain size as it will be seen later, and the lagoon barriers were not very hospitable areas; their development and drain improvement have allowed the touristic use (6,13), which is the most intensive one now. Some coastal works for harbour facilities generated artificial beaches of some quality which, though not foreseen at the time,
can be perfectly explained now, with the present theoretical knowledge about the littoral dynamic and processes (4,5,11, 12,13). Both the morphology and the variable orientation of the coastal stretch studied introduce some variety in the general processes, in a similar way to the ones described in a previous work (8). In the following analysis several previous studies about the available wind regimes for these coasts have been taken into account. (7).

II. LITTORAL DINAMIC

No appropriate and sufficient directly registered or observed data of the sea exist to define the wind wave regime for Santa Pala bay. Data of the World Meteorological Center are scarce and "Ocean Waves Statistics" Tables (9) are imprecise (7) for them. An acceptable way is based on the forecasting methods arising from wind regimes.

The general atmospheric circulation on East-Southeastern Spanish coasts is conditioned by the following extensive factors (7): a) the polar front; b) the North-Atlantic (500) extratropical low pressure centers (cyclones) migrating from West East up to its dissipation over the European plain; c) the high-pressured North-Atlantic (300 about) areas; d) the strong continental character of the Peninsula; e) the seasonal and very mobile Ligur center of low pressures, and f) the desertic character of North-Africa, with its also mobile cyclonic area. Of all of them, factor b) is the origin of the strongest winds, though its influence is rather lower in Mediterranean than in Canarian and Atlantic coasts; in the former the Northern extratropical cyclones cause some important NE winds only in the few occasions they reach central Europe; the southern cyclones, whichever its origine and typology, may produce lower, though not irrelevant, winds from SE and E. Besides we must have in mind the breezes caused by thermic gradient over the marine - continental interphase.

For a more detailed study of the winds in this stretch, the records of the closest weather stations have been analysed. They are strongly affected by breezes, because of their situation in the interphase, just as we have been able to verify comparing them to the Britain's Admiralty Meteorological Charts (2,7). Two of the observational areas of these charts may be considered suitable for that checking: one onto the sea of Oran, at the Southeast of the bay, and another inland onto the interior of the Peninsula, around the coastal zone; besides, the former is perfectly available to give us the swell-marine wind regimes to use in wave forecasting determinations for Santa Pala bay (7). Breezes and other local winds can affect littoral processes acting over dunes and barks but they have scarce influence in the littoral transport (7,8). However swell winds have an important influence in them through generating waves.

During the autumn (October) winds are weak and from all directions but SE; this exception becomes more outstanding and spreads out to S and, less pronounced, to E during the winter (November-February), when the influence of the fourth quadrant (littoral transport from the NW
to the SE in this stretch) grows up. During the springtime (March-May) there begins a change establishing a very noticeable alternative influence of "Levantes" (from E and, in a minor degree, from NE) and "Ponientes" (from W and, less, SW), and this situation goes on during the summer when "Levantes" become progressively dominant. During all these months, from March on, the N and NW winds are decaying to become so irrelevant as their opponents, the S and SE winds. West winds are noticeable in berm nourishing, lagoon-filling and onshore and offshore flattening processes. The latter is the first condition to permit some new barrier in these coasts, but the lack of important wave storms from the fourth quadrant avoids it.

The relative importance of the East and Northeast winds cannot impose a net longshore transport all along the bay. As a matter of fact the wind waves reaching the shore are reduced—because of the fetch configuration coefficient (7, 11)—and rather affected by the refraction and diffraction processes, which make them to attack the shore with a minor angle. For all these reasons the North to South dynamic (actually East-West) only have prevalence under the cape, up to the port. At the South of the port the shoreline orientation and the reasons above balance somehow both longshore transport directions. Nevertheless this transport is irrelevant in relationship with the relative weakness of the fourth quadrant wind and waves.

Both dynamics are interrupted by the port producing, at the North, the "Levante" beach, and at the SW, a very noticeable flattening of the bottoms and a reduction of the depths in a wide band, which as a matter of fact avoids the waves reaching the shoreline. At the same time, this moves backwards as a consequence of a loosing of steepness. That bottom situation is available to permit the formation of a new barrier from the secondary port breakwater to the SW up to the zone in which the longshore dynamic is the smallest.

In relationship with the Levante beach, though the port cannot be considered as a totally efficient transversal barrier, the hypothesis of a nourishment from the South is all negligible because of the orientation of the shore and the relationship between the breakwater and the cape sizes and arrangement. The materials passing in front of the breakwater Northwards avoid the cape and reach the "Arenales del Sol" beach. And the hypothesis of a nourishment from the North, though admissible, only may be applied to materials proceeding of the erosion of the cape or, in a minor degree, passing in front of the port from the South. The cape's size and morphology reinforce other general longshore transport materials from raising the shore.

Though Santa Pola bay is highly sheltered some wind wave storms have attacked its coasts and produced appreciable damages as in 1927-28 or in 1980-81, principally in the stretch between the cape and the port. In those occasions the "Levante" beach seesaws supported by the port principal breakwater up to lose part of its sands which pass southwards in front of it. Such a process may take the sands to the entrance channel and to proper harbour.
III. DESCRIPTION AND GEOMORPHOLOGY (10)

Betic chains direct their alignments from the Peninsular South to "San Antonio" and "La Nao" capes and faraway under the sea emerging again in Pitiusas and Mallorcan Isles. They are bordered along their Southeast side, in the province of Alicante, by Neogenic and Quaternary sediments which characterize, more or less, these littoral areas. Pre-betic and Subbetic chains, at the North, and the actual Betic ones, at the South, close a wide oval shaping the coast between "La Nao" and "Palos" capes. Santa Pola Bay is in its middle zone. The wide, high montainous arc, close to the coast, and other minor sierras and spurs reaching the sea, have been the feeder of materials forming the several plains.

Next to the coastal area studied, important carbonated Jurassic and Cretaceous Subbetic formations are over other Mesozoic and Low-Tertiary prebetic ones. The latter can be easily characterized, but the former offer many difficulties because of their strong tectonic in nappe and scales. The actual Betic outcrops and formations reach Callosa and Orihuela sierras, at the NE, bordering the left side of the low stretch of the Segura river, whose alluvial plain must be partially laid on them. They are Triassic limestones, dolomites and phyllites, and Permian slates and quartzites. All these formations are bordered in their east side by vast Quaternary sediments which spread out up to the shore, between two Tertiary spurs reaching the sea: the sierra of Santa Pola, at the North, and the northern foothills of the sierra of Cristo, at the South of Guardamar; into this Quaternary plain outcrops the also Tertiary sierra of the Molar. The Quaternary leans to the Mesozoic through different Tertiary lengthy bands with a betic (SW-NE) direction. These bands lie discordant on older deposits and mark, probably, the different positions of a shoreline which have suffered several transgression-regression processes. The dominant materials are the Mesozoic's after their several removals and posterior sedimentations, as both Tertiary and Quaternary limestones, loams and other debris.

Lately, these different formations have suffered more or less acute erosive processes, depending on weather conditions, up to the present morphology, in which the Quaternary deposits forming the ending plains of Vinalopó and Segura rivers are especially important. The Vinalopó and the Segura rivers, and other torrents and "ramblas" have canalized the generally intermittent run off of the different mountain alignments closing the Elche-Santa Pola plain. All of them have transported important sediments towards the coast, favoured by both their steep slopes and the lithology.

Therefore three geological ensembles define this area: the Betic alignments, the small and more or less disperse Tertiary sierras, and the more extensive Quaternary plains. The interaction of these elements have conditioned the evolution of these coasts into a variable weather situations and a certain littoral dynamic, on the basis of a progressive advance of the shoreline since the high-Tertiary. The process, not yet definitively explained, cannot be considered a systematic and continuous one.
but it is in accordance with the Quaternary weather variations and the corresponding sea level fluctuations, and it is besides in relationship with a tectonic factor. Being this factor the final sequence of the Alpin orogenesis, which still remains in the high seismicity of this zone (Sangonera-low Segura falla and Torrevieja-San Miguel line). Without entering in the discussion about the subsidence or the emergence of this coastal band, the Vinalopó-Segura plain seems to correspond to some process based on several successive barrier and lagoon formations. The Hondo and Hondo Amboz pools and the several sab-pans of Santa Pola hold up this hypothesis. The "bonifications" of 17\textsuperscript{th} century and on in the fertile low plains of the Segura and the Vinalopó rivers, the control of run off for irrigations and the unschedule actions over the freatics, and the very recent massive building are the several antropic factors which have conditioned the last evolution of this coast.

At present, the general shoreline arrangement, from La Nao to Palos capes, marks an approximately average direction NE-SW, in several minor arcs forming a great oval. The North arcs have a preferential E-W direction but the South ones follow a preferential N-S one, as it is the case with Santa Pola bay. Santa Pola coasts form the Northern part of this bay and have two fairly different parts (fig. 1). The Northern one, from the village, is much defined by the Santa Pola sierra and cape, which spreads out to the Quaternary and recent deposits such as an old Quaternary isle, incorporated to the mainland by the progressive advance of the shoreline. This advance possibly happened as a consequence of several barrier formation processes followed by the posterior filling of the lagoons. The coast of the cape is not a plain but a rough though low one, where a few double-supported small beaches exist, and whose limestone rocky shoreline is linked to the Tertiary old levels by characteristic removal "glacis". In the Southern end of this part two wider beaches have appeared, supported on two breakwaters which were built.

VII.251
for harbour facilities.

At the South of the port and the village, up to the Segura mouth, a wide sandy formation has been developed forming the plain coastal limit, whose shoreline changes its orientation from E-W to adapt itself to the cape, to N-S. From South to North four different beaches are comprehended into one only sandy formation: the "Pínet", the "Eras del Port", the "Playa Lissa", and the "Gran Playa", with one unique singularity affecting the two former ones and formed by the Vinalopó delta front. All of them have a very flat slope, very fine sands, dunes behind the beach and a fair stability, but the more we go towards the port, the flatter the slope is, the finer the sands are, there are fewer dunes, and the more the shoreline seems to regress.

The sources of materials for these beaches are of three different nature: a) the erosion of the cape; b) the general longshore transport; and c) the Segura and Vinalopó contributions. As the shoreline geometry shows, with both deltas (4,13) eroded, the present contribution of both rivers is scarce, though in the past it was a very important factor as it has been shown. The complete exploitation system of their waters is the cause of it.

IV. DISCUSSION AND CONCLUSIONS

For this study several different works and researches have been developed over one year, to recognize and filter seasonal changes. Though no abundant chartographic and topographic documents are available, it has been possible to analyse the plans of the different harbour designs (1906 and 1933), which show the Levante beach, and the five aerial photographs (1947, 1956, 1966, 1972 and 1977), which have permitted the respective restitutions of the shoreline from the Levante beach towards the South (3) and show the coastal arrangement based on both deltaic and barrier processes. Eleven point, P-1 to P-11 (fig. 1) have been sampled several times along the year at several levels, getting different sand samples from the dunes, beach, foreshore and nearshore zones. Their grain size analysis show some variations in relationship with all three following factors: point of the coast (P-1 to P-11), distance to the shore and depth. The first factor also affects the nature of the grain and depends on both material sources and exposure degree to waves. The second factor acts in close relationship with longshore transport circumstances, affected by the relative size and positions of both the harbour and the cape and by the littoral morphology. The third factor shows the dependence on the changes in both wave energy (short term) and nature and origin of sediments (long term). Nature analysis has only looked for the rate of the most abundant fractions and the roundness degree of their grains. The distribution of the less frequent heavy minerals had been studied (1) through all Valencian coasts and in this work only quartz -white, grey and red-, micas -biotite and moscovite-, tourmaline, circon and lime organic rests have been considered.

Onshore-offshore profiles have shown in some points the swell storm effect,
(fig. 2), in spite of the highly flattened bottoms, the bars moving offsho-
rewards and the shoreline moving land-
wards. The profiles have been determi-
ned three times: 19.06.80, 23.10.80 and
19.02.81. Between the second and the
third times some important storms happe-
ned. Most of the beach and berm of the
cape-coves desappeared (see P-8) and
the profile of the "Levante" beach (P-3)
showes a backwards movement of its shore-
line point due to the storms. Neverthe-
less P-1 and P-2 profiles show an advan-
ce of their shoreline points in accor-
dance with an adequate combination of
winds, swell waves and sea-level varia-
tions (5,13) acting on a very flattened
beach; they extracted important quanti-
ties of sand towards backsoreh all
along "Playa Lissa" and "Gran Playa"
beaches. The advance of the shoreline
point in P-5 and P-6 was caused by the
"Varadero" beach accretion with mate-
rials from the cape, the former, and by
the shelter produced by the L groins,
the latter. The seasonal variation of
the plans (fig. 3) of the "Varadero"
and "Levante" supported beaches was
appreciable, and they were caused by a
seesawing along their respective sup-
ports. In the "Levante" beach some
sands have even passed southeastwards
in front of the tip of the breakwater.

The aerial photographs show two di-
fferent evolutions of the shoreline po-
sition at the South of the port. All
the Southern stretch of beaches, with
a N-S orientation, have suffered some
alternant movements, with a 25 meters
net average advance since 1947 to 1977,
though such a general process is modi-
fied in several singular points (little
quays and inlets, and Vinalopó mouth),
which have suffered no or little regres-
sions. These and other beach varia-
tions of a shorter period (1947-1956;
1956-1965; 1965-1972; and 1972-1977),
have been likely caused by seasonal
transversal profile changes, which have
been previously analysed, or may be ex-
plained by the sea level at the moment
of being photographed, apart from the lo-
cal influence of the singularities (12).
The shoreline forms around them show a
prevalent S to N little longshore trans-
port. Recently, from 1972, the more
northwards (from the "Pinet" to "Bras-
del Port" beaches) it is observed the
more noticeable some erosion is.

In the Northern stretch of beaches,
close to the port, with an approximate
E-W orientation, 100 meters average sho-
reline regression is noticed since 1947,
though most of it has occurred between
1947 and 1956 (75%). But at the same
time a process of bottom flattening and
onshore zone widening has been occurring,
as it may be known by the recent burial
of a longshore submerged rigid shoal or
barrier, which might be a fossil quater-
nary bar. It produced an additional
shelter to the old Santa Pola haven.

Therefore, in spite of the shoreline re-
gression of this stretch it should not
be inferred that a coastal regression was
being produced but, perhaps, a comple-
tly different process of coastal advance
of which the present phase would be the
preparatory flattening of the nearshore.
It would avoid the high power wave
components to reach the shoreline and
to produce wide profile variations. On
the other hand, the great width of the
onshore zone makes more effective the
refraction phenomena reducing the long shore transport. Only S to SE wind waves would be able to modify the present profile but an adequate high sea level and wind strength would be necessary. Nevertheless provided these very infrequent conditions took place it would be possible the formation and remaining of an emerged barrier closing a new shoal, littoral lagoon in this stretch. The importance of the present harbour breakwaters (1944-1960) may have been noticeable cutting the local E to W long-shore dynamic.

Between those two different stretches a gradual change of circumstances and variation trends is produced approximately along the "Lissa" beach. The principal factor in this transition is the change of shoreline orientation. Finally, the complexity of the different local shoreline movements from 1947 to 1980, all along the coast at the South of the port, may suppose that some kind of hyperannual cyclic nature of the processes exists. In fact, it is possible to suspect that there have been some important storms in the 1947-56, 1956-65 and 1972-77 periods.

Eastwards of the port only variations in "Levante" and "Varadero" beaches are discernible. The latter has continuously accreted and it seems to have reached some kind of limit because its growing rhythm has diminished from 1972; nevertheless that limit may be originated by the lack of materials or by the loss of efficacy caused by the important damages of the groin; in this last hypothesis it would be possible to expect some extraordinary growing up of the "Levante" beach in the next future. The accretion of this beach was not so continuous and it has suffered frequent seensaws (fig. 3). Hyperannual cycles and the Varadero breakwater secondary effect could be the responsible factors of regressions before 1947. The recent accretion in the 1947-72 period is in relationship with the damages in the Varadero breakwater; lately it may have started to lose sand in front of the
port and, if so, the seesaw effects are irreversible. Recently the port had to be dragged for the first time. A second factor increasing the seesaw has been the construction of a vertical wall for defense in the east end of the beach.

Some sieve analysis (figure 4) show the local influence of the Vinalopó (P-1.6) within the general characteristics of the sands in this coast (likely from the Segura delta, P-2.a; P-1. 50 m.) (a stands for the surface sample of the beach of the respective profile; b, c, stand for the sample of different depths of the beach; 50 m. stands for the sample of the bottom at about 50 meters from the shoreline; and P.n, stands for the profile). The inactivity of the Vinalopó river is shown in the transitional aspect of the P-1 a. and P-1. dune curves. The sieve analysis of the samples from the little cape beaches (figure 5) shows the unity of provenance. The figure 6 shows the differences of the samples from the same level along the shoreline: P-1. a. shows the influence of the Vinalopó materials, while P-5. a., P-6. a. and, in a minor degree, P-3. a. show the one of the cape erosion over the Segura sands (P-2.a. and P-1. 50 m.). Finally, the influence of the storms and their power is shown in fig. 7.

The results of the mineralogical analysis of the samples have shown the importance of the different quartzes (fig. 8), while only traces of micas have been found. Samples from P-6 and P-7, and some of the P-2 and P-5, show the influence of the groins and port breakwaters quarrystones erosion (to notice, particularly, the proportion of ciron). The influence of the materials from Segura river may be shown in a minor proportion of white quartz and a bigger one of grey quartz. The reduction of lime organic rests in some samples is in relationship with the shelter degree, but their fraction is less important than in other parts of these Peninsula coast, in any case. The nearly total lack of micas in all the samples can be especially meaningful in bearing out the hypothesis of the very important role of the Ebro materials in the construction of the Oval Valencian coasts at the North of San Antonio cape. A meaningful fraction of micas—especially of biotite—has been found in our researches there (4), while they are missing at the South of San Antonio cape.

From the previous discussion and the comprehensive analysis of all aforementioned researches it has been possible to propose the following statements.

1.- The littoral dynamic into the Santa Pola bay is highly conditioned by the Fetch configuration and the refraction and diffraction phenomena, which modify the primary winds action, particularly the dominant one which is the action of the first quadrant winds: the distant presence of the Balear Isles and La Nao cape and the immediate protection of the Tabarca Isle and Santa Pola cape means an important Fetch configuration coefficient, the latter Isle and cape cause a noticeable diffraction and the flatness and width of the coastal platform produce a strong refraction; all three factors together reduce noticeably the wave energy reaching the shoreline from those directions. Winds coming from the other directions are ra
Fig. 8
ther less important. As a consequence the littoral dynamic is not very strong.
2.- The longshore component of that littoral dynamic is only substantial on shore from the cape to the port and it directs permanently westwards because of the particular E-W orientation of that stretch. At the South of the harbour that longshore component is scarce and alternating and its net annual value must be directed towards the North (passing in front of both the port and the cape) all along the bay; but it is much diminished in the near onshore, particularly next to the harbour where haven and morphology practically avoid all littoral dynamic.
3.- More important is the transversal component of the littoral dynamic though it only has a certain impact capability on the cape; at the South, it is insufficient to modify the present flatness of the onshore and offshore bottoms. This is just the adequate situation that may permit the formation of a new littoral barrier if some important wave storm with an appropriate transversal component appears. Lately the dialectical relationship between the shoreline direction, the coastal arrangement and the littoral dynamic was leading to minimize the littoral processes, and only the cape behaves as both a singularity and a support. At present, the port has separated the instable (at the Northeast) and the stable -or hyperstable- (at the South) stretches. If there were enough materials for it, the last destiny of the cape (and of the port) would be its inclusion into the coastal formation, losing its singularity in the new shoreline.
4.- It has been shown the influence of the materials of Vinalopó and Segura rivers in the arrangements of this coast and in the formation of its beaches. Though the importance of each one has changed along the time, Segura river has been prevalent. Local materials proceeding from the cape erosion abound in the Varadero and Levante beaches stretch, but it is caused by its isolation discussed above. So it must be noticeable in this coast the effect of the systematic reduction of the continental materials reaching the sea. For this reason the coastal formation processes are now slower or even about to stop, as a general rule, and in this case it is difficult to forecast if the shoreline will go on advancing by forming forward a new barrier. Nevertheless, except in some points that will be discussed later it is not possible to state, at least for the moment, that this is a regressive coast. The different beaches and sub-stretches show the following variations:
a) "Pinet", "Braun del Port" and "Lissa" beaches up to the change of the orientation of the shoreline have not shown any noticeable regression, though some wasting of sands is produced by wind action and dune movement and dissipation. The principal though scarce materials supply is from the South, from the Segura and Vinalopó deltas which are in erosion processes. These sources make up for the present inactivity of their rivers.
b) "Playa Lissa" beach, from the change of the shoreline orientation, and "Gran Playas" beach have suffered
some shoreline regression along with a notorious flattening of the bottom which cannot be considered as an erosion process. On the contrary some accretion of materials is admissible and it would be suitable to permit a natural or artificial littoral barrier.

c) The renewal of materials in the "Levante" beach is rather difficult because both the "Varadero" and the harbour breakwaters behave as transversal barriers. The wasting of sands is produced after a seesawing process caused by the dominant waves and supported on the port breakwater. This seesawing is hardly reversible because of the protection of the very breakwaters to the northwards dynamic.

d) The cape coasts are under erosion processes and only in some coves offer little double-supported (inserted) and stable beaches though they respond strongly to E and NE swell waves sometimes disappearing most of their berms after the winter storms.

V. RECOMMENDATIONS TO ACT

In accordance with the discussion of the previous chapter the possibilities of implementation are different for the different stretches. Any action along the cape shoreline is possible if the scale, form and arrangement are suitable. Between Varadero and harbour breakwaters several implementations are suitable in order to improve the ambivalent and touristic capabilities of the area, but some of them appear to be urgent to keep the "Levante" beach and the promenade so that some designs have been proposed as the one below. Finally, at the South of the harbour, though not urgently, other totally different kind of implementations are recommended.

In order to stabilize the "Levante" beach, protecting at the same time the explanade beside, four different elements have been proposed to be built (fig. 9).

![Fig. 9](image)

Three of them are groins and the fourth is a shore revetment with an alignment more parallel to the wave crests than the present shoreline. The groin (a) is to avoid the sands passing in front of the harbour, but it produces a differential shelter and, if it were the only one, the seesawing of the beach would be more noticeable. Therefore the b and c groins are to fix the plan of the beach; their size and distance are in relationship and they are bigger than the suitable ones for the present size of the beach, but the artificial nourishment is planned, taking sands particularly from between a and b groins. The revetment is to avoid the shoreline erosion without too much wave reflection; looking for a more suitable alignment the explanade has been enlarged to improve the recreative facilities.

In order to improve the ambiental quality of the "Gran Playa" beach offe-
ring some new recreative possibilities the artificial implementation of a littoral barrier has been proposed. Such a solution may produce no or very few negative impacts and it is a very "natural" one because it is a quasi-reproduction, in advance and short time, of the foreseeable evolution of the shore.

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MICROSTRUCTURAL FEATURES OF THE MARINE SOILS IN CHINA

CARACTERES DE MICROSTRUCTURE DU SOL MARITIME CHINOIS

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ABSTRACT

The microstructure of marine soils from the East China Sea, the South China Sea, the Yellow Sea and the Bohai Sea has been studied with scanning electron microscope, so as to establish the relationship between the engineering properties and the microstructure. It has been found that the microstructure of marine soils is complex and variable, it is closely related both to the deposition environment and its engineering properties. Based on their features, a system of classification of the microstructure of these soils has been proposed. Different types of the microstructure have different engineering properties. Hence, it is possible to estimate the engineering properties of marine soils according to the proposed classification.

I. INTRODUCTION

The engineering properties and microstructure of marine soils have been studied rather early in several countries. In China, some institutions have also studied the engineering properties of marine soils, but no study on microstructure has ever been attempted. The author with the help of others has recently collected marine soil...
samples from different sea areas in China (see table 1) so as to study their microstructure, and established the relationship between the microstructure and the engineering properties of marine soils.

Table 1: Physicomechanic Properties of Marine Soil Samples

<table>
<thead>
<tr>
<th>Sea area</th>
<th>Location of sampling</th>
<th>Depth (m)</th>
<th>Moisture content(%)</th>
<th>Void ratio</th>
<th>Liquid limit(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>West Bohai Tianjin</td>
<td>2-3</td>
<td>44.9-57.9</td>
<td>1.32-1.59</td>
<td>38.8-41.9</td>
<td></td>
</tr>
<tr>
<td>West Bohai Tianjin</td>
<td>20-24</td>
<td>25.5</td>
<td>0.72-0.74</td>
<td>30.1-31.3</td>
<td></td>
</tr>
<tr>
<td>North Bohai Liaodong gulf</td>
<td>8-10</td>
<td>38-45</td>
<td>1.04-1.27</td>
<td>29-36</td>
<td></td>
</tr>
<tr>
<td>North Bohai Liaodong gulf</td>
<td>28-30</td>
<td>21-25</td>
<td>0.601-0.697</td>
<td>23-28</td>
<td></td>
</tr>
<tr>
<td>Middle Bohai Bohai gulf</td>
<td>33</td>
<td>29-32</td>
<td>0.0848-0.0966</td>
<td>32.9</td>
<td></td>
</tr>
<tr>
<td>Middle Bohai Bohai gulf</td>
<td>64</td>
<td>23-24</td>
<td>0.065-0.0672</td>
<td>27.1</td>
<td></td>
</tr>
<tr>
<td>Yellow Sea Dalian</td>
<td>3.5</td>
<td></td>
<td>1.50</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>East Sea Shanghai</td>
<td>26</td>
<td></td>
<td>4.74</td>
<td>34</td>
<td></td>
</tr>
<tr>
<td>South Sea Zhongjiang</td>
<td>16</td>
<td></td>
<td>4.17-1.28</td>
<td>65.7</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Plastic limit (%)</th>
<th>Clay content &lt;5µ(%)</th>
<th>Compression coefficient a1-2</th>
<th>Sensitivity St</th>
<th>Allowable bearing capacity (Kg/cm²)</th>
<th>Shear strength (d A°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>8</td>
<td>9</td>
<td>10</td>
<td>11</td>
<td>12</td>
</tr>
<tr>
<td>19.3-20.6</td>
<td>0.058-0.063</td>
<td>5.2</td>
<td>0.50</td>
<td>0.12</td>
<td>4</td>
</tr>
<tr>
<td>22.7-23.9</td>
<td>0.010</td>
<td>3.2</td>
<td>1.85</td>
<td>0.10</td>
<td>27</td>
</tr>
<tr>
<td>16-20</td>
<td>0.103</td>
<td>3.7</td>
<td>0.80</td>
<td>0.09</td>
<td>16.5</td>
</tr>
<tr>
<td>16-20</td>
<td>0.107</td>
<td>3.7</td>
<td>2.25</td>
<td>0.39</td>
<td>37.5</td>
</tr>
<tr>
<td>20</td>
<td>1.62</td>
<td>1</td>
<td>0.21</td>
<td>5.6</td>
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<tr>
<td>14</td>
<td>15</td>
<td>5.32</td>
<td>0.08</td>
<td>8.5</td>
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<tr>
<td>20</td>
<td>15-35</td>
<td>2.5</td>
<td>0.08</td>
<td>8.5</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>47-52</td>
<td>10</td>
<td>0.41</td>
<td>8.5</td>
<td></td>
</tr>
</tbody>
</table>

*: Apparent Preconsolidation Pressure

The samples collected were taken from representative strata at different depths. The East Sea samples were taken from a greyish-green hard clay stratum which is common in the coastal area of Shanghai. It has good engineering properties and is the load bearing stratum for piles under tall buildings and inshore structures.

The samples from Bohai Sea include those from Tianjin–Xingang area and others from the North and Middle Bohai. Some of them are sludge clay from shallow depths while others are samples from the load bearing layer for inshore structures at greater depth.

The Yellow Sea samples are also sludge clay from shallow layers deposited newly in the offshore area near Dalian, containing a lot of diatomite. Their engineering properties are poor.

The South Sea samples are very sensitive clay taken from the coastal area of Zhongjiang. This is a peculiar kind of clay, its physical index seems poor but its mechanical properties are not so bad.

The number of samples collected is not large, but they are representative of the marine soils encountered in the coastal region of China.

They have been examined with scanning electron microscope of the British type S4-10 and the Dutch type PSEM-500K. Micrographs of various magnification (from 40x to 10000X) have been taken. Energy dispersive analysis of x-ray has also been carried out to establish the mineral composition of some constituents. Observations have shown that the microstructure of marine soils is complex and variable, it is closely related both to the deposition environment and its engineering properties.

II. MICROSTRUCTURE FEATURES OF MARINE SOILS

1. The form of grains of soils

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The samples are all clayey soils containing a great deal of clay colloidal particles and a certain amount of silt and sand grains. Also contained are many fine bioremais, diatomites and various micro-crystals of salt. They have a certain influence on the engineering properties of marine soils. The forms of particles and contained-matter observed will be discussed in the following.

(1) Forms of clay colloidal particles

The clay colloidal particles are generally flat platelet with a size of 1–3µ and a thickness of about 0.1µ. It rarely exists alone; usually several or even tens of platelets aggregate together forming 'domain' usually flat, which go into the making of the basic micro-fabric units (Plate I, Photo 1–2). They form further into aggregation of various sizes with diameters of about 10µ–80µ, becoming a matrix of soil mass.

(2) Forms of silt and aggregate grains

The silt grains are mineral fragments smaller than the sand grains while aggregate-grains consisting of fine particles cemented together are similar to silt grains in size. Since the silt grains are usually coated with mud and fine particles on their surface, it is difficult to distinguish between silt grains and a aggregate grains from their appearance (Plate I, Photo 3–4). In general, these aggregate grains and silt grains suspending in clay matrix do not contact each other (Plate I, Photo 5). However, in the Bohai Sea, they come in contact with each other, and almost no clay particles are seen at the contact points (Plate I, Photo 6). This is due probably to the fact that the composition matter has been originated from the Loess Plateau and Huabei plain.

(3) Forms of micro-crystals

A variety of micro-crystals exist in the sea. Some of them are salt crystals formed when the water content of soils decrease and they stick on the grain surface (Plate I, Photo 7–8). Some aggregate together forming matrix of structure mass (Plate I, Photo 9–10). Others are formed with bio-remains intergrowth (pyrite), forming complete crystal (Plate I, Photo 11–12). These different forms of crystal and their mode of occurrence have a certain influence on the strength of the soil structure.

(4) Forms of bio-remains and diatomites

A great number of bio-remains and diatomites exist in the sea (Plate II, Photo 1–2). The diatomites have various forms, such as spheroidal (Plate II, Photo 3), disc (Plate II, Photo 4), honeycomb (Plate II, Photo 5), rings (Plate II, Photo 6) as well as saucers etc. In some sludge clays, the content of diatomites is very high and they may be easily broken due to low strength (Plate II, Photo 7). Therefore, the existence of diatomites in the soil mass structure has a certain influence on engineering properties.

2. The pores of marine soils

It is generally considered that high porosity is an important structural feature of all marine soils. In fact this is not always true. From the soils that have been studied, it has been found that as the depth increases, void ratio decreases from 1.25 to 0.65, and the density of the soil obviously increases. Evidently, the void ratio is associated with the size, shape and mode of occurrence of the pores that exist in the structure of soil mass. According to their shape and size, pores may be divided into three categories:

(1) Macropore

Macropore exists in soils at shallow depth. Its size, on the order of 100–300µ, is much larger than the diameter of grains (or aggregates). The pores penetrate through the whole soil mass (Plate II, Photo 8). It is similar to the macropores of loess soil. The difference between them lies in the fact that the grains around macropores in loess are usually strongly cemented by calcium carbonate, while the cementation of particles around macropores of marine soils is much

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weaker (2) (3). Therefore, under pressure the grains may easily move, and the pore collapses, hence the soil mass deforms.

(2) The intergranular pore

This kind of pores exist between silt grains and aggregate grains or aggregation. The amount of intergranular pores is the main factor which determines the porosity and deformability of the soil mass. Loose soils from shallow layers have larger and more intergranular pores (Plate II, Photo 9-10) while those from the dense or deep layers have smaller and less (Plate II, Photo 6).

(3) The intragranular pore

It is the pore that exists within aggregate grains or aggregations. Generally, the pores within aggregate grains only affect the porosity and exert no influence on the deformation property of the soil mass (Plate II, Photo 11-12), but some loose aggregations having intragranular pores to such an amount that they may also affect the deformation property of the soil mass under external pressures (Plate I, Photo 1).

3. The cementation bond and connector bond between skeleton grains

The term 'skeleton grains' here refers to those soil grains that transmit stresses in the soil mass under external pressures. They act as skeleton. Some of them may be very large or very small, some are 'domain' of clay, and others may be fragment grains or micro fragment aggregation. However, there are some grains existing within the macropores or intergranular pores; they act as stuffing (Plate I, Photo 1, Plate III, Photo 1), transmitting no forces. With regard to the cementation bond and connector bond of skeleton grains, this kind of stuffing grains is not included. Therefore, in making observations, skeleton grains should be distinguished first with low magnifying micrograph, and then analyzed with high magnifying micrograph.

Cementation bond and connector bond produced between the skeleton grains connect the disperse grains together, so that they can sustain external load with out producing relative displacement and deformation of the soil mass. However, the strength of some bonds may be very low; in such case under small external load the connected points may break and slip, resulting in deformation of soil mass. Therefore, to analyze and study the strength of the cementation bond and connector bond between skeleton grains is an important aspect for distinguishing the engineering properties of marine soils.

The cementation bond or connector bond of marine soils is very complex, it varies with different geological conditions and geographical environments. The cementation materials and the modes of bond are different. The following types of bonds have been observed:

(1) Salt-crystal cementation bonds

The salt content in sea water is very high, the salt content of the fluid in the pores of marine soils is still higher. With continuous increase of salt concentration, the salts of low solubility crystallize first and separate out, sticking to the surface and connected points of grains in micro-crystal form (Plate III, Photo 2-3), acting as cementing agents. The strength of salt crystal cementation changes as the salt concentration of the pore water changes.

(2) Clay cementation bonds

Clay cementation generally occurs between aggregate grains and silt grains coated with clay. The surface of grains are covered with a thick layer of clay platelets. As the grains contact each other, the clay platelets act as cementing agents (Plate III, Photo 4). This kind of cementation generally has very high strength so as to enhance the stability of soil mass.

(3) Connector (link, bridge) bonds

Connector bonds generally occur between aggregate-grains and aggregations consisting of clay particles or clay domains in the marine water. The large assemblages (includes aggregate grains and aggregations) having a certain interval do not contact each other.

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Some of the small assemblages may act as links, pulling up two larger assemblages, while others act as bridges, joining large assemblages together (Plate III, Photo 5-7). The strength of these bonds is usually low, depending on the length to breadth ratio of the connecting links, the larger the ratio, the weaker is the strength. Under continuous shear stress, creep deformation will occur.

III. TYPES OF THE MICROSTRUCTURE OF MARINE SOILS

It is generally believed that the microstructure of marine soils, which has been formed by micro-particles deposited under the marine water conditions, has a honeycomb structure and a spongy structure. In fact, the microstructure of marine soils is very complex and variable. According to the different features that have been observed, the microstructure of marine soils may be divided into the following types, namely: granular cementation structure, granular link-bond structure, flocculent link-bond structure and the clay particle matrix structure. They may be subdivided further.

1. Granular cementation structure

In this type, the aggregate-grains or silt grains act as skeleton grains which basically contact each other, so the intergranular pores are small. This kind of structure usually exist in soil layers below ten meters in the Bohai and Yellow Sea inshore at intervals, having small void ratio (0.6-0.8) and low compressibility (Plate III, Photo 2-3). This structure may be subdivided into two subkinds, depending on the cementing material, as the granular salt-crystal cementation structure and the granular clay cementation structure.

2. Granular link-bond structure

When the aggregate-grains or silt grains act as the skeleton grains, there are certain intervals between them and they are joined together by link consisting of clay domains. The soil structure with large intergranular pores.

(Plate III, Photo 6-7, Plate II, Photo 9) is generally found in shallow layered soils in the Yellow Sea and Bohai Sea. According to the length of the links, it may again be classified as the granular long-link bond structure (Plate III, Photo 6) and the granular short-link bond structure (Plate I, Photo 6).

3. Flocculent link-bond structure

In this type, the floccules act as skeleton grains and they are joined together by links or 'domains' of clay, forming flocculent link-bond structure. This kind of structure is similar to the flocculated structure that has been discussed in textbooks (Plate I, Photo 1-2). It exists generally in the sludge clay no deeper than six meters in all sea areas in China. On basis of the density of floccules and the length of bond links, it may be subdivided into four subkinds: the dense flocculent long-link structure, the dense flocculent short-link structure, the open flocculent long-link structure and the open flocculent short-link structure.

4. Clay matrix structure

Clay matrix structure consists of a great deal of clay colloidal particles. Clay 'domains' conglomerate to form regular or irregular aggregations and then conglomerate further to form large clay ped.

The outline of the aggregations may be distinguished in some clay pedds, while in others indistinguishable (Plate III, Photo 8). Occasionally, some silt or sand grains are enclosed in the clay matrix, forming what may be called the clay particle matrix structure. If the arrangement of 'domain' or clay platelets within the clay matrix is dense and in parallel (face-face lamina), the micropores are small and few, it is called the oriented clay particle matrix structure (Plate III, Photo 9). If the arrangement of 'domain' or clay platelets within the aggregation is loose and at random (edge-edge angle bond type), the micropores within are large and many and interconnected with each other in open arrangement (Plate III, Photo 10),
it is called the open particle matrix structure. This kind of microstructure has been found in all sea areas in China.

From the foregoing, the different types and features of microstructure of marine soils may be summed up in Table 2.

<table>
<thead>
<tr>
<th>Grain form</th>
<th>Bond mode</th>
<th>Classification of microstructure</th>
<th>Subkind of microstructure</th>
</tr>
</thead>
<tbody>
<tr>
<td>aggregate or silt grain</td>
<td>cementation</td>
<td>salt crystal cementation structure</td>
<td>granular saltry crystal cementation structure</td>
</tr>
<tr>
<td>aggregate or silt grain</td>
<td>link bond</td>
<td>long granular link-bond structure</td>
<td>granular clay particle cementation structure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>short</td>
<td>granular short-link bond structure</td>
</tr>
<tr>
<td>dense</td>
<td>link bond</td>
<td>long floculent link-bond structure</td>
<td>dense floculent long-link structure</td>
</tr>
<tr>
<td>open</td>
<td></td>
<td>short</td>
<td>dense floculent short-link structure</td>
</tr>
<tr>
<td>clay particle matrix</td>
<td>dense</td>
<td>no link clay particle matrix structure</td>
<td>clay particle oriented matrix structure</td>
</tr>
<tr>
<td>open</td>
<td></td>
<td></td>
<td>clay particle open matrix structure</td>
</tr>
</tbody>
</table>

IV. THE MICROSTRUCTURE AND ENGINEERING PROPERTIES OF MARINE SOILS

In the twenties, Terzaghi related the engineering properties of marine soils to their structure, and suggested the honeycomb model to explain their mechanical properties. Later, Casagrande supplemented Terzaghi's model and suggested a complex model containing silt and sand grains to describe the sensitive structure of marine deposits. Recently, with the development of scanning electron microscopy, the microstructure of marine soils have been studied extensively. Many new concepts and terms have been proposed to substitute the old ones based on the concept of single clay particles. These concepts and terms have systematically been summed up by K. Collins and A. McGown (1), and have been included in the book 'Soil Properties and Behaviour' by R. N. Yong (6).

However, traditional habits usually people misconceptions. It seems that the particular engineering properties of marine soils such as high sensitivity, high porosity, high rheological effect, high compressibility and low strength are all due to a single kind of floculent honeycomb structure which exist in all marine soil layers. Obviously this is a misunderstanding. Engineering experience has shown that even in the same sea area under the same condition, in the same drilling hole at same location, the difference between the engineering properties of the shallow layer soil and the deep layer soil may be very large.

As stated previously, the microstructure of marine soils is complex. By comparing the microstructure features with the results of physicomechanical tests, it is evident that the engineering properties are closely related to the type of microstructure, for example: High compressibility. It generally occurs in floculent open
long-link structure. The 'long-link' indicates the existence of a great deal of unstable intergranular pores; the 'open' show that has many intragranular pores. Soils with this kind of structure will undergo large deformations under pressure.

High rheological effect. It primarily occurs in long-link structure. Regardless of the skeleton grains being granular or flocculent, soils with this type of structure will undergo long-time flow deformation under stress. According to R. Pusch (5) and R. Ohta (4), this phenomena is due to the tension and distortion of links caused by shearing stresses (Plate III, Photo 7).

High sensitivity. The sensitivity of marine soils depends on the mode of arrangement of the 'domains'. If the clay domains have edge-face-angle space net arrangement (Plate III, Photo 11-12), it possesses certain space rigidity, but when this arrangement is destroyed the strength will quickly decrease, thus showing high sensitivity. The granular structures of salt-crystal cementation and short-link bond also manifest a certain degree of sensitivity.

Low strength. The strength of marine soils is not always low. Granular cementation structure and clay particle oriented matrix structure may have high strength. Some of them may be used as the load bearing stratum for piles.

The relationship between various types of microstructure and the engineering properties are listed in Table 3 for reference.

Table 3: Engineering Characteristics of Marine Soils with Different Microstructure

<table>
<thead>
<tr>
<th>Types of structure</th>
<th>Engineering characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength</td>
</tr>
<tr>
<td>Granular salt-cementation structure</td>
<td>medium to high</td>
</tr>
<tr>
<td>Granular clay particle cementation structure</td>
<td>medium to high</td>
</tr>
<tr>
<td>Granular long-link bond structure</td>
<td>low</td>
</tr>
<tr>
<td>Granular short-link bond structure</td>
<td>medium to low</td>
</tr>
<tr>
<td>Dense flocculent long-link bond structure</td>
<td>medium to low</td>
</tr>
<tr>
<td>Dense flocculent short-link bond structure</td>
<td>medium</td>
</tr>
<tr>
<td>Open flocculent long-link bond structure</td>
<td>very</td>
</tr>
<tr>
<td>Open flocculent short-link bond structure</td>
<td>low</td>
</tr>
<tr>
<td>Oriented clay particle matrix structure</td>
<td>medium to high</td>
</tr>
<tr>
<td>Open flocculent clay particle matrix structure</td>
<td>medium to low</td>
</tr>
</tbody>
</table>

V. CONCLUSIONS

1. The microstructure of marine soils in China is complex and variable. Its formation relates not only to the geological conditions, buried depth and the environment of deposition, but also relates to the origin of the composition matter.

2. A classification of the types of microstructure has been proposed for the marine soils in China.
according to the structural features. Based on this classification and physico-mechanical test data, it is evident that the engineering properties of marine soils are closely related to their microstructure.

3. It is expected that on basis of further studies on microstructures, the engineering properties of marine soils may be reasonably estimated from their kind of structure.

Acknowledgements: The Author is grateful to Messrs. Gu Dezhen, Chen Zhongyi, Zhang Xiangong, Zhang Zong-hu, Chang Shibiao for reading the manuscript, and thanks the units and individuals who supplied the samples and test data.

REFERENCES


Plate I

1. Clay-domain form x5000 (Yellow Sea)
2. Clay-domain form x10000 (East Sea)
3. Aggregate grain form x10000 (Bohai Sea)
4. Aggregate grain form x5000 (Bohai Sea)
5. Aggregate grains suspending in clay matrix x5000 (East Sea)
6. Aggregate grains contact each other x1250 (Bohai Sea)
7. Salt crystals sticking at grain surface x640 (Bohai Sea)
8. Salt crystals sticking at grain surface x1250 (Bohai Sea)
9. Salt crystals matrix x1250 (Bohai Sea)
10. Salt crystals matrix x5000 (Bohai Sea)
11. Pyrite crystals form x1250 (Yellow Sea)
12. Pyrite crystals form x5000 (Yellow Sea)
ACOUSTICAL AND ENGINEERING PROPERTIES OF TRIAXIALLY LOADED OFF-SHORE SOILS FROM BOMBAY HIGH

LES PROPRIETES ACOUSTIQUE ET MECANIQUE DES SOLS AU LARGE DE HAUT-FOND BOMBAY EFFECTUEES DANS L'APPAREIL TROIS-AXIELS

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ABSTRACT

The recent activities in oceans have brought experts from various disciplines on a common platform to work together for the successful completion of various types of near off-shore and off-shore projects. Consequently, the need to interpret the data of one discipline in terms of the complementary discipline has arisen. One such pair of disciplines which have greater possibility of coordination are Geophysics and Geotechnique. The authors have developed an Ultrasonic Monitoring Triaxial Apparatus 'UMTA' for the simultaneous study of acoustical and certain geotechnical properties of triaxially loaded soils. The present paper gives brief details of the 'UMTA', and the results of tests conducted with it on undisturbed off-shore soil samples obtained through the Oil and Natural Gas Commission from Bombay High area in India. Measured acoustical P-wave velocities of the samples were found to be indicative of the soil type, and stress history.

ABSTRAIT.

Les activités récentes dans la mer ont été la raison de travailler ensemble de beaucoup de spécialistes des études différents pour le succès des projets varié au large et à terre. Il devient, donc, nécessaire qu'on puisse interpréter les données d'une division de l'étude par les données d'autre division de l'étude. Deux divisions telles qui ont plus de champ d'action en cette direction sont la geophysique et la geotechnique. Les auteurs ont développé un appareil ultrasonique, s'appelle 'Ultrasonic Monitoring Triaxial Apparatus (UMTA)', pour l'étude simultanée de la propriété acoustique et quelques propriétés mécaniques des spécimens unmodifiés du sol qui ont été éprouvé dans l'appareil trois-axiaux. La présente communication commence par décrire brièvement les détails de l'appareil UMTA et expose les résultats des études expérimental effectuées dans cette appareil sur les spécimens unmodifiés du sol au large obtenus de haut-fond Bombay (Bombay High) par le Commission de l'huile et de gaz naturel (Oil and Natural Gas Commission). Il s'est trouvé que la calérité mesuré de l'onde P des sols est un indicateur de la type du sol et la pression qui existait antérieurement dans le sol.

INTRODUCTION

The evaluation of geotechnical properties of soils through the acoustical wave propagation properties and acoustical emission
characteristics determined in-situ requires an inter-disciplinary approach. Such an indirect method could be useful to geotechnical engineers for field and design activities executed with speed and economy with special reference to near-off-shore and off-shore struc-
tures. The method also combines the advantages of remote monitor-
ing and non-destructive testing. The monitoring of compaction and
deforation of earthen structures non-destructively are specialized
tough allied fields of research (Koerner 1978, Freeme 1978, Singh
and Jakhamwal 1982a). The simpler of these correlations of acoustical
P-wave propagation properties of soils with their geotechnical pro-
erties has also been attempted recently by the authors (1982b).
The work in this direction was started by the authors with the
development of two ultrasonic apparatuses, which facilitated the
simultaneous measurement of acous-
tical P-wave propagation properties and certain conventional geotechni-
cal properties of the uniaxially and triaxially loaded soils. These
apparatuses have been named Ultrasonic Monitoring Consolida-
meter 'UMC' and Ultrasonic Moni-
toring Triaxial Apparatus 'UMTA' for
soil testing (Singh and Jakhamwal
1980, 1981b). The initial findings
by the authors (1981c) showing that
the acoustical P-wave propagation
properties of uniaxially loaded
soils depend upon the soil type,
void ratio, degree of saturation
and stress history of the soil,
corroborated the earlier findings of several authors, notably Biot
(1956), Gasman (1951) and Toksoz
(1976). These studies were both
theoretical and experimental in
nature. A direct application of
the acoustical (geophysical) test
results obtained in 'UMC' and
'UMTA' was considered more suitable
for off-shore geotechnical problems
because of the fact that the off-
shore soils are basically a two
phase medium comprised of soil
solids and water completely fill-
ing the pores. This makes the soil
simpler from the acoustical wave
propagation point of view, making
the correlation of geotechnical
properties with geophysical
property theoretically sound. This
was the basis for motivating the
authors to conduct tests on off-
shore soil samples with 'UMTA'.
Undisturbed off-shore soil samples
were obtained from Bombay High
through the courtesy of Oil and
Natural Gas Commission, Bombay Off-
shore Project (India).

**THE APPARATUS**

The special features of the
Ultrasonic Monitoring Triaxial
Apparatus 'UMTA' are shown in
Fig. 1. Two special ultrasonic
probes, one at the bottom and the
other at the top were used for
transmitting and receiving the
ultrasonic waves. The bottom
probe contains the drainage gallery
which has an outlet at its bottom.
This outlet makes the drained as
well as undrained tests under back
pressure with pore pressure meas-
urements possible. The top probe
is of hollow chrome steel. The
chrome steel material for this
purpose helps in reducing the fric-
tion between the sleeve of the
triaxial cell and the stem of the
probe, which works as a loading
piston also. The bottom probe is
made of brass. The copper tube
used for drainage cum back pressure
was soldered to the top of the
probe. The conventional details of
the drainage gallery, pore cum back
pressure tube and the triaxial
apparatus are available in litera-
ture (Bishop and Henkel, 1957).

The special features of the
apparatus, including conventional
and newly developed facilities are
summarized:

A. Type of triaxial apparatus:
   Stress controlled

B. Independently variable para-
parameters:
   (i) axial stress,
   (ii) lateral pressure,
   (iii) back pressure.

C. Measuring facilities of for
   dependent variables:
   (i) vertical strain,
   (ii) volume change,
   (iii) pore pressure,
   (iv) ultrasonic time lag.

D. Type of crystals used:
   Lead zirconate titanate
   matched crystals of 160 KHz
Fig. 1 ULTRASONIC MONITORING TRIAXIAL APPARATUS 'UNTA'
natural frequency.

E. Propagation technique used: Through propagation technique.

F. Electronic device: Pulse-generator-cum-oscilloscope giving 250 PRR for exciting the transmitting crystal with the facility of measuring time-delay upto an accuracy of 0.1 micro-second.

G. Pre-amplification circuitry: Fabrication of a pre-amplification circuitry was required for boosting the weak signals coming from the receiving probes (Singh, Kak and Jhakhawal 1981d).

CALIBRATION OF THE INSTRUMENT

The apparatus needed two types of calibration, viz. (i) mechanical calibration and (ii) time-lag calibration. The method of doing these calibrations is given below:

(1) Mechanical calibration: The all-round lateral pressure on the sample was applied by means of constant pressure mercury control system. The increase in cell pressure brings about an uplift pressure on the top probe. In order to keep the top probe just in contact with the sample, calibration tests were required to determine the counterbalancing loads on the piston for different cell pressures. A dial gauge was used for this purpose. At each cell pressure, the counterbalancing load was increased till a frictionless movement was noticed. Fig 2 shows the required calibration. This calibration is required once in the beginning for each apparatus, and the one present here pertains to the apparatus developed and used by the authors for applying the same all-round pressure on the sample while ensuring monolithic contact with the top probe with the perspex block placed at the top of the block and these calibrated counter-balancing loads were applied on the piston through a frame as per the requirement during experimentation. This ensured the correct measurement of time-lag.

(ii) Time-lag calibration: A porous stone/perforated perspex disc with a filter paper was used above the bottom probe, over which the sample rests as shown in Fig. 1. A perspex block was positioned on the top of the sample. The top probe was made to touch this perspex block by applying counterbalancing loads for time-lag measurements as mentioned above. The time-lag thus measured gave the delay time of the total system including the sample. A time-lag calibration of the system for the necessary correction was therefore required before conducting the tests. This calibration was performed by using standard materials, e.g., aluminium and sand in place of the soil sample. First of all the time-lag through the standard aluminium sample was measured separately in the normal way. Next the total time-lag for the standard sample positioned in the 'UMTA' was measured. The time-lag calibration through the system only (without sample) was then obtained by deducting the time-lag for the sample only from the total time-lag observed. Every time a soil sample is tested, the calibration time so determined is deducted from the observed time-lag, for finding the time-lag through the sample.

TEST PROCEDURE

The undisturbed off-shore soil sample was extracted in the standard triaxial mould from the bulk sample obtained through the Oil & Natural Gas Commission, Bombay Offshore Project, Bombay (India). The sample was then placed in the 'UMTA' as shown in Fig. 1. It was provided with filter paper side drainage for permitting easy drainage of pore water during drained condition under each applied all-round pressure. The total time-lag for the acoustic waves to travel from one end of the sample to the other was determined by means of an oscilloscope-cum-pulse-generator by observing the time-lag of the first arrived signal coming to the receiving probe. A dial gauge was used to measure the changes in the height of the sample during the application of loads. The net time-lag was determined by deducting the
Fig. 2. Calibration curve
time-lag calibration from the observed total time lag, and then the acoustical wave velocity through the samples was calculated.

RESULTS AND DISCUSSION

Table I shows the geographical location, depth, Atterberg classification and density of the soil sample.

The variation in acoustical P-wave velocity of the soil samples with the increase in back pressure under zero effective pressure conditions is shown in Fig. 3. It is evident from the figure that in case of off-shore soils complete saturation of the soil sample occurs at a back pressure of 0.5 kg/cm². This pressure is considerably less than that observed for laboratory prepared samples obtained by gradual consolidation starting from the slurry stage, and which was found to vary between 1.0 kg/cm² and 1.25 kg/cm² (Singh and Jakhanwal 1961a). This indicates that the off-shore soil samples possessed a higher degree of saturation than the laboratory prepared samples.

The results of the further tests conducted for studying the variation in acoustical P-wave velocity of the soil samples under different effective pressures are shown in Fig. 4. For these studies, the back pressure was kept constant at 1.5 kg/cm² and the cell pressure was increased in steps of 1.0 kg/cm² to 4.5 kg/cm², so as to achieve effective pressures of 1.0, 2.0, and 3.0 kg/cm², respectively.

The acoustical P-wave velocity values obtained conform broadly to the range of values reported for ocean sediments by Hamilton et al. (1956). The values of acoustical P-wave velocity at zero effective pressure are more dispersed for MI-OI group of soils than for the CI group, and they tend to converge at higher effective pressures. In the case of ocean sediments like Bombay High, the stress history effect can be considered to be directly related to the depth of sediment for the same type of soil deposit. Hence, it can be concluded that the stress history effect is best noticed in such off-shore deposits in the condition of zero effective pressure testing. This stress history effect tends to become insignificant at higher effective pressures of testing, corresponding very nearly to the pressure of overburden for the particular type of soil sediment.

The distinct nature of the curves for CI and MI-OI group suggests that there is a marked effect of the soil sediment type on the acoustical velocity values measured through them at different effective pressures. However, similar tests on many more off-shore soils are needed to confirm the above findings.

<table>
<thead>
<tr>
<th>Platform</th>
<th>Sample No.</th>
<th>Depth (ft)</th>
<th>Depth (m)</th>
<th>Geographical Location</th>
<th>Density (gm/cm³)</th>
<th>Atterberg classification</th>
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<tr>
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<td></td>
<td></td>
<td></td>
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<td>Longitude</td>
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<tr>
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Fig. 3. Acoustical velocity versus pressure (under zero effective pressure condition).
Fig. 4. Acoustical velocity versus effective pressure (keeping constant back pressure of 1.5 kg/cm²)
CONCLUSION

The off-shore soils tested from the Bombay High were found to be of CI and MI-OI groups. The tests conducted in the newly developed Ultrasonic Monitoring Triaxial Apparatus 'UMTA' show that the acoustical P-wave velocity of these soils depends upon the soil type, stress history, and the saturation condition of the sample. The effect of the soil sediment type is quite pronounced covering the entire range of testing pressures, and the effect of the stress history which is pronounced at zero effective pressure tends to become insignificant at higher effective pressures.

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