Seminar on Geotechnical Engineering in River Basin Development

Organised by the

Sri Lankan Geotechnical Society

in association with

Sri Lanka National Committee on Large Dams

3rd December 1993

Foreword

The Sri Lankan Geotechnical Society and the Sri Lanka National Committee on Large Dams have jointly organised the Seminar on Geotechnical Engineering in River Basin Development. Ten papers covering different aspects of Geotechnical Engineering associated with the development of river basins are presented in this volume.

Since water is one of the most important natural resource of this country, high priority has been given for the development of Irrigation and Hydro-power schemes. Several such projects involving the design and construction of dams, spillways, tunnels, power houses and irrigation channels have been completed and several more are still in the planing and investigation stage. This Seminar is being organised with a view to record and disseminate the vast experience gained by our Engineers and Geologists in the planning, design and construction of such schemes.

It is very important that the engineers in this country should keep abreast with the development taking place in the engineering profession internationally. Otherwise, this country will have to depend on foreign consultants who will not be familiar with the geotechnical conditions in this country, to advise on engineering development projects. Hence, the engineering profession and professional societies associated with it should encourage and ensure that advanced knowledge and experience available be disseminated to all the members of the engineering profession.

A. Thurairajah
President
Sri Lanka Geotechnical Society
3rd December 1993.

Message from the President - SLNCOLD

Today's Seminar on "GEOTECHNICALENGINEERINGINRIVERBASINDEVELOPMENT" is the second in the series organised by the Sri Lanka Geotechnical Society and the Sri Lanka National Committee on Large Dams. The first in the series titled "DESIGN AND CONSTRUCTION OF DAMS"was held on 14th March 1987 and sponsored by the Central Engineering Consultancy Bureau.

Professionals getting together at seminars of this nature are important because knowledge is exchanged and even more important short comings and mistakes made in the past can be discussed openly and in a truely technical sense.

In the development of future hydropower resources Sri Lanka, it may perhaps be no longer possible to resort to traditional methods because of the high cost involved and alternative methods have to be considered with a view of reducing cost.

I strongly feel that we have to develop our strategies and go in for long term development with minimum foreign aid like the staged development adopted for the development on the Kelani Ganga. There is no difficultyin undertaking these Projects ourselves iffinance is made available as there is sufficient local expertise to carry out there Projects.

We have also to think in terms of cost saving approaches like for example the use of rollcrete. This has a very big potential in our country with the possibility of reducing construction costs.

I hope that the papers presented today willprovoke new thinking in technology suitable to Sri Lanka in the present times.

I wish the seminar all success.

VIDYAJOTHIDR. A N S KULASINGHE

SEMINAR
ON
GEOTECHNICAL ENGINEERING IN RIVER BASIN DEVELOPMENT

VENUE : Sri Lankan Association for Advancement of Science, Colombo 07

DATE : 3rd December 1993

A G E N D A

8.30	9.00	Registration
9.00	9.05	Welcome address – Vice President SLNCOLD
9.05	9.15	Address – President S.L.G.S.
9.15	9.25	President SLNCOLD

SESSION I

Chairman – Dr. A.N.S. Kulasinghe

9.25	9.45	River Basin Development – N. Madusudanan
9.45	10.05	Geotechnical Engineering in the Design of Dams – Ms. R. L. Hathurusinha
10.05	10.25	Problems encountered in Dam Foundations & Remedial Measures – S.H.C de Silva
10.30	11.00	Tea

SESSION II

Chairman – Prof. A. Thurairajah

11.00	11.20	Dispersive characteristics of soils used in the construction of Earth dams and Embankments – Ms. V. Kumarasamy
11.20	11.40	Instrumentation in River Basin Development – Dr. Sunil de Silva
11.40	12.00	Guidelines for Design of Pressure Tunnels – Dr. G.P. Rajapakse
12.00	12.45	Discussions
12.45	13.30	Lunch

SESSION III

Chairman – Prof. B.L. Tennekoon

13.30	14.00	A Case Study of Samananalawewa Dam and Reservoir Leakage – Mr. Vernon Perera
14.00	14.20	Problems encountered in Tunnel Construction (Case Histories) – A.K.D.N. Atukorala
14.20	14.40	Reservoir water tightness – R.L. de S. Munasinghe
14.40	15.00	Geodetic Surveys in Construction of Dams and Tunnels – T. Somasekaram
15.00	15.30	Discussions
15.30	15.40	Summing up
15.40	15.45	Vote of Thanks – Secretary SLGS.
15.45	16.15	Tea

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RIVER BASIN DEVELOPMENT

by
Eng. N. Madusuthanan

Today we are at a seminar on Geotechnical Engineering in River Basin Development. It will illustrate the important roles of Geotechnical engineering and the Geotechnical engineer in developing river basins. As many of the audience may be new to river basin development, I hope to introduce what river basin development means.

Rivers are copious natural streams of water flowing in channels to the sea or lakes. In Sri Lanka we have 103 rivers. These originate in the central hills and flow down the valleys and ultimately into the sea. Their source of water is rainfall.

Let us imagine that Sri lanka is an absolutely flat land and that it receives our requirement of water for drinking, washing and plant growth by the required rain falling daily, in the nights when we are comfortably sleeping. Being a flat land, the rain water will spread evenly and there will be no rivers. As it rains the required amount daily, no structures are required to store or divert water. All the development which requires water will take place without any Geotechnical, Irrigation, Water Supply or Drainage engineer. Only hydro power will not be available. There will be no floods or droughts.

If now we imagine that Sri Lanka is hilly (as it is now), but the rain falls daily as imagined earlier, then there will be small perennial rivers. All development, including electricity can take place without storage. There will be no floods or droughts. We engineers are not required.

Coming back to reality, we find that rainfall is never uniform in nature. It is a very random occurrence. It falls at irregular rates and in irregular amounts. It can be of short duration (say 5 minutes) or of long duration (say 5 days). When we take long duration rains, in most instances there are periods during this duration where there is no rain at all. From studies of observed rainfall data over a number of years, we are able to prepare total rainfall curves and rainfall intensity curves for different duration of rains (or storms) of various frequencies. The rainfall intensity is the total rainfall divided by the duration of the storm. The instantaneous intensity during this duration may vary widely.

In Sri Lanka, the rainfall pattern is such that we can identify two distinct zones called the Dry Zone and the Wet Zone. At the boundary we have the intermediate zone. In the Wet Zone the rainfall is sufficiently uniform for plants to grow without (or with little) irrigation from stored water. There are a number of diversion schemes in this Zone. In this Zone the rainfall exceeds the evapo-transpiration of the plants. The boundary closely follows the 2,000 mm isohyet. In the Dry Zone irrigation from stored water is a must for successful cultivation of paddy and many other crops.

Due to the irregular nature of rainfall, the river flows themselves are irregular. During dry spells the river may be a small trickle even be dry. During wet spells the river will be in floods. The amount of water carried by the river at any point depends on the catchment (or watershed) area at that point, the amount and extent of rainfall experienced in the catchment for the last few days, or weeks, the slopes, soils and landuse of the catchment amongst many other factors. The watershed area of the river is known as the river basin.

River basin development is the art and science of developing this basin so that its resources (primarily water) is made to benefit mankind before it finally discharges into the sea. This would mean storing water during the wet spells for use during the drier periods. Excess water manifests itself as floods. Such waters may have to be removed fast to prevent damage or diverted to areas where there is a shortage of water. When water is diverted to other basins it is known as trans-basin diversion. Sometimes the excess water may be pumped up to higher levels and stored for subsequent use during dry weather.

Water being an essential requirement, historically civilisations developed in the river valleys. Thus ancient civilisations developed around Nile, Indus and other large valleys. In every country the civilisation developed near the river banks. Even now we see the civilisation around Mahaweli Ganga, Kelani Ganga, Kalu Ganga and other rivers. In spite of the frequent threat of floods, the people return to the river banks once the floods recede. Controlling the rivers to reduce flood damage, provide water for human and animal use, increase agriculture and produce electricity are some of the aspects of river basin development.

Such development invariably requires many structures to store or divert water or generate electricity. These structures have foundations. They affect the ground conditions and stability in the neigbourhood. Many of these structures have to be water tight. Hence we see that Geotechnics, Geotechnical engineering and Geotechnical engineers are necessary and vital elements in river basin development.

Now let me go from the general to the specific case of the studies required for river basin development. When a river basin has to be developed, a large amount of data collection for hydrologic, hydraulic, geologic, soils, land use, socio-economic and other studies are involved. A number of disciplines have to work to-gether as a team. Studying a full basin will involve a system study where water will be transferred between reservoirs and even between basins as optimally required. This is quite complex. For our illustration we will consider how a single storage reservoir is designed.

The hydrologic study will first take into account the rainfall pattern in the basin. Then storage or diversion points are selected along the river to study the development potential at that point. Such points have to be also topographically and geologically suitable for a structure. In the preliminary study this may be established only by visual inspection and study of available maps till the hydrologic study establishes the type and size of the structures required.

Two of the hydrologic studies undertaken for determining the type and size of the structures are: Operation Studies and Flood Studies. Let me explain each of these studies in outline.

It is necessary to determine the optimum capacity of storage reservoir required to cultivate the land within the command of the reservoir, or to generate firm hydropower or both. For this we use the rainfall and, if available, riverflow data. The longer the number of years for which the data is available the better. If only rainfall data is available, then we estimate the yield (the amount of water coming into the reservoir) by using methods which establishes the relationship between rainfall and run-off. Then the demand of water required by the crops and/or electricity users are determined. For the crops this would mean the water requirement for land preparation and by the crop at various stages of its growth. For electricity generation, this would mean the study of the pattern and amount of electricity required for domestic and industrial use. Then we assume a capacity for the reservoir and that it is in operation from the year for which rainfall values are available. Then a month by month water balance is performed on the system and check if the required demand of water can be over the years of study. Normally we do not try to get 100% success. This would be too costly and in any case the rainfall pattern is not going to repeat in the future. This is a statistical assessment and we try to achieve 85% success in the operation study. Then other capacities are tried out so that we can optimise the design. If available land is a limitation then we try to cultivate the full extent using the minimum storage. If storage is a limit, then we try to command the maximum extent using the largest practical storage.

The water balance equation we use is of the form:
Storage at beginning of month + Inflow - Losses - Demand - Spillage
= Storage at the end of the month
= Storage at the beginning of the next month
The main losses taken into account are seepage and evaporation.

In the Irrigation Department, a large number of small reservoirs were designed and built under various projects such as Village Irrigation Rehabilitation Project (VIRP). To enable such designs to be done rapidly computer programs were developed to do this operation study. But these tanks were sited in areas where no rainfall stations were available nearby. Even if they were available the effort of entering the data of a number of years and the amount of calculations involved would not justify its use for such small tanks in large numbers. Hence we adopted a 75% probability rainfall pattern over a single year and an operation study performed over a year. The answers were found to be realistic and a manual was prepared on using this method including methods for flood studies. The world bank also approved the use of this method for small projects.

Now let us see what Flood Studies are. While the operation studies determine the optimum capacity of the reservoir and extent to be cultivated and/or electricity that can be generated, Flood studies determine the size of spillways required. During times of heavy rains, water in excess of what can be stored has to be safely led off back to the river. This is done by the construction spillways. For flood studies, we use rainfall intensity-duration-return period graphs and flood hydrographs. We have a few basins where the actual river flows have been measured. For flood flows the measurements of the river flow when in spate is very important. However, measurement of riverflow under flood conditions is dangerous and difficult and in most stations these critical measurements are not available. However, now new techniques are being adopted to take these measurements. Hence certain amount of extrapolation and sound judgement has to be exercised in the use of these values. For smaller catchments we use synthetic hydrographs using Run-off coefficients arrived at using a combination of experience, measurements and sound judgement.

For small catchments Irrigation Department has developed computer programs which studies the effect of rainfalls of different durations in steps of one minute, and determines the critical duration of the storm which will cause the highest discharge through the spillways. For flood studies we consider the reservoir to be full before the onset of the storm which is a conservative assumption. When the tank starts spilling, the water level rises thus holding back a certain amount of water known as the detention volume temporarily. This helps to keep the outflow discharge lower than if there is no detention. Thus we have to make a compromise between increasing the detention and thus submerging more land above the full supply level of the tank and increasing the spillway size.

When we develop the water resources potential of a river basin, we have to consider it as an integrated project where one element (say a reservoir) in the catchment affects another element (another reservoir). Sometimes we develop a catchment by constructing a major reservoir which requires almost the whole water resource potential of that catchment for successful operation. Subsequently if we build a large number of small tanks in the catchment of the larger reservoir, the large reservoir will suffer due to lack of water. Also the overall benefit in the basin will be reduced. Hence when a basin or valley is developed it is necessary for a simple authority to b vested with necessary power to develop the catchment as an integrated system. Then only we can reap the best benefit from the river basin development.

In the design of the structures such as Dams, Spillways, sluices, powerhouses and channel systems the Geotechnical engineer plays an important role. All the above structures have to be stable under various conditions of loading, the bearing capacities and settlement of the foundations have to be properly studied and designed.

I hope that this introduction to river basin development and the important role the Geotechnical engineer has to play in the river basin development will help to appreciate every discipline and the parts they play in a river basin development project.

Thank you very much.

GEOTECHNICAL ENGINEERING IN THE DESIGN OF DAMS

R. L. Haturusinha

1. INTRODUCTION

The engineering of dams plays a vital role in the story of civilization. Dams have been linked closely to the rise and decline of civilizations, especially to those cultures highly dependent upon irrigation.

Man has paid great attention to develop and utilize water resources, since the nature supplies water at cyclic intervals which do not coincide with his need. Large dams and storage reservoirs have been built across many streams to control water levels and to store water in period of abundance to be used during periods of low river yields.

Concurrently, there have been advances in related technologies which have significantly influenced all types of dams. Important among these are engineering economics, geology, hydrology, seismology and computational techniques. Other sciences have exerted influence on particular dam types. This is well illustrated by the impact of geotechnical sciences on embankment dams, although the impact of these sciences on foundation for all types of dams has been significant.

The purpose of the associated project has a major influence on the type of dam selected to be designed and built. The earliest dams to be built were earth dams which were followed by rock-fill dams. Masonry dams then followed and concrete dams came considerably later when Portland Cement became sufficiently plentiful and economical to allow faster construction and more watertight structures. Improvements in rock-fills continued greatly aided by improved quarrying methods, placement practices, membranes and compaction. The use of earth embankments continued when site conditions were favourable.

The selection of a suitable dam type depends on vicinity conditions, geology of alternative sites, topography, hydrology of the streams, available materials and economics of the project which the dam is to serve.

As the use of dams has expanded, there has been a large expansion in the scope and the use of engineering geology. Many early dams were sited with little more geological examination than inspection, experienced judgement, topographic mapping and a few diamond drill borings. Seismic concern was minimal at this stage. The need for detailed geological examination of dam foundations and burrow materials were strongly emphasized by the failures of dams. Detailed geologic mapping, extensive use of exploratory trenches, adits & tunnels have become common practice.

During recent years instrumentations to measure earthquake forces has yielded a large database on seismic magnitudes, accelerations and, movements, which is now used in dam designs. There is no conclusive way of predicting reservoir induced seismicity, or further, of conclusively interpreting causes if such a condition is suspected.

More attention has been focused on reservoir-geology during recent years. Potential slide areas are carefully explored and searches are made for solution channels and other information which might produce subterranean outlets. Reservoir rim conditions which might cause leaks are carefully explored and reinforcements are made as may prove necessary.

2. SELECTION OF DAM TYPE

The objective of planning is to locate, design and construct most economical and sound structures to achieve the purpose of the associated project. It is therefore useful to have an idea of various types of dams, factors that govern the selection of site and selection of a particular type of dam for a given site.

2.1 TYPES OF DAMS

Earth-fill Dams can be constructed on earth, as well as on rock foundation whereas other type require more stringent foundation conditions. Their construction involves utilization of great variety of locally available materials in their natural state with a minimum of processing. They generally are competitive in cost with concrete and rock-fill dams.

Rock-fill Dams require less severe foundation conditions than for a concrete gravity dam and more severe than for an earth-fill dam. Bedrock foundations which are hard and erosion resistant are the most desirable for rock-fill dams. The use of foundations consisting of river gravel or rock fragments is acceptable.

Concrete dams may be classified as gravity, arch or buttress according to their design.

Concrete gravity dams by their own weight resist forces imposed on them with a desired factor of safety. They are adapted to sites where there is reasonably sound rock foundation, although low structures may be founded on alluvial foundation if adequate cut offs are provided.

Concrete arch dams are curved in plan and transmit water load primarily by means of thrust to the abutments, there by utilizing the compressive strength of the abutment material. Abutments should be composed of good rock to resist the end thrust.

Concrete buttress dams have a component that retains the reservoir, usually a flat slab, which is supported by a series of buttresses. The slab is inclined in the downstream direction. The thrust from the reservoir and the dead load of the slab are transmitted to the buttresses and thence to the foundation. The foundation requirements are not very much different from that of a concrete gravity dam.

2.2 Factors Affecting the Choice of Dam site

The selection of suitable site for a dam depends on many factors. But, in this paper only the geotechnical aspects are considered.

Foundation

The site should preferably have a good sound rock for foundation. For a concrete dam, solid rock at the surface or within a reasonable depth below it, is essential. For arch dams strong abutments are essential.

Rock-fill and earth dams have more flexible foundation requirements, but even then, the cost of the dam would be materially affected by the type of foundation available.

Sediment load

The sediment load in the stream should be as minimum as possible to assure that the dam would not silt up within a short period after impounding.

Spillway Site

A part of the dam is usually designed to act as a spillway in the case of concrete dams. However, for earth and rock-fill dams, the spillway must be located separately. The best site is that in which the gorge portion is separated from the flank.

Availability of Construction Materials

It is economically feasible if bulk of the material required for the dam is available in close vicinity of the site. Earth dams can be designed to utilise almost any type of materials available at site, though the economy of the work is affected accordingly.

Diversion during construction

Sometimes river diversion problems play an important role in the selection of dam site. This factor may affect design of the dam and also the construction schedule. Common practice for diverting streams during construction utilizes one or combination of the following provisions.

* tunnels driven through the abutments
* conduits through or under the dam
* temporary channels through the dam
* multiple stage diversion over the alternate construction blocks of a concrete dam.

Submergence

The stability of the reservoir slopes due to submergence must be taken into account in selecting a dam site.

Water Tightness of Reservoir

The rim of the reservoir should be watertight at least up to the proposed elevation of the dam. The stored water should not be able to escape under the surrounding hills through cavernous rock or other continuous pervious strata.

Access Conditions

The construction of access roads in hilly terrain requires geotechnical investigation for the stability of constructed roadways and the material through which the roads have to be formed. Bad site conditions may affect the economics adversely.

2.3 Selection of a suitable type of Dam

The selection of dam type best suited to a particular site is purely a matter of judgement and experience. However, an intelligent study of the existing conditions and requirements will assist in the selection considerably.

The choice of dam type is directly influenced by the natural conditions of the site such as topography, geology, hydrology and seismic activity and by local conditions such as availability and accessibility of construction materials and transport facilities. It is also indirectly influenced by the purpose and size of the dam, as well as by the construction time and methods and the labour, materials and machinery required. These in turn are influenced by safety, legal, aesthetic and economic consideration and social conditions.

Therefore, it is only in exceptional cases that an experienced designer can say that only one particular type of dam is suitable or most economical for a given site.

Some of the physical factors that influence the selection of dam type are considered below.

Topographical characteristics such as the cross-sectional profile of the canyon and the contours of both abutments of the dam site influence the choice of dam type. The former has an important bearing on the required volume of material in the dam while the latter mainly affects the stability of the dam.

Main types of Valleys are as follows :

* Gorges	Chord-Height ratio under 3
* Narrow Valleys	Chord-Height ratio 3-6
* Wide Valleys	Chord-Height ratio more than 6 or 7

A thin arch dam is more suitable for a gorge. But the rock in the gorge should be capable of withstanding high pressures and should not fail by shearing. However, thick arch dams have been built in narrow valleys having abutments with sound rock. Concrete gravity dams are also suitable for narrow valleys with sound foundation conditions and weak abutments. In a wide valley practically every type of dam can be constructed except a single (thick or thin) arch. In such a valley, the type of dam is governed primarily by geology of the site and the availability of construction material at the vicinity.

Geological features that affect the selection of dam type are:

* Quantity and quality of overburden and river depth
* Strength and uniformity of the foundation material, the geological character and thickness of strata,their inclination, and relation to underlying strata, existing faults and fissures.
* Water tightness of the foundation material.

The foundation material will limit the choice of type to a certain extent, although such limitations will frequently be modified, considering the height of the proposed dam.

Influence of earthquakes on design criteria varies widely from country to country, depending on seismic activity. Probably, the type of dam least vulnerable to earthquake is a properly designed fill type dam. Next comes the gravity type dam.

These factors which affect the choice of dam type are so closely connected and so complicated that it is necessary to depend upon a sound engineering judgement for economy and safety of the dam.

3. INVESTIGATIONS

Investigations are planned and carried out to permit analysis of alternative proposals so that a final plan can be formulated to obtain maximum benefit from the available resources and achieve multiplicity of purpose. Before commencing investigations, the purpose, scope, and type of the project must be defined and it must also be ensured that the project does not conflict with other existing or contemplated projects. The investigations should be adequate to safeguard against premature failures or unforseen problems during course of construction and operation and maintenance.

Investigations, if carried out to completion, are costly and time consuming. With reference to all types of investigations required for dam construction, the code of Practice of the Institution of Civil Engineers recommend that up to 7 per cent of the cost of the structure is permissible for site investigation. But, the usual practice is to limit it to 1 per cent of the total cost of the development. By nature, geological investigations for completion of information are very expensive and time consuming. The accuracy of information sought depends considerably on the magnitude of the proposed project as well.

Investigations are carried out to ascertain whether a project is economically or technically sound and are divided mainly into three stages, each identified by its objective.

* Reconnaissance
* Feasibility
* Detailed Design

Investigations may be necessary even after formulation of final plan. They could, continue during construction period as well as during the operation and maintenance period.

3.1 Reconnaissance

The purpose of reconnaissance stage of investigation is to determine rough feasibility of the project. The geological data collected in this stage are primarily descriptive and only preliminary investigations are carried out in this stage. Data available are summarized and evaluated and further supplemented with rough additional data. Visual inspection of the site and surrounding area is also made. The field work is done with the assistance of a competent Geologist.

The data obtained during reconnaissance stage are used in preliminary designs and estimates to evaluate engineering and economic possibilities of the project to define its physical limits, and to determine whether detailed investigation should be proceeded with. Such designs and estimates are used as an aid in selecting the most economical plan when several alternative possibilities exist.

During reconnaissance stage, the geological investigations should lead to an appraisal of the general sub-surface conditions throughout the area of the project as well as an evaluation of the broad aspects of the foundation conditions of the alternative sites selected. The appraisal should define major advantages and defects of the foundations and material deposits at the alternative sites with reasonable certainty. The most desirable site may be investigated further by additional surface examination and a limited amount of sub- surface exploration and testing.

Reconnaissance stage investigations will lead to preparation of the preliminary project report for the selected site.

The **preliminary investigations** usually require,

* A not too precise study of site survey with the resulting small topographic site map having contours defined by a minimum of controlling points or sketched from the cross-sections.

* Some investigations of the overburden

* A few borings in consultation with a geologist according to the magnitude of the project and character of the foundation.

* A preliminary geological investigation and report for the site, borrow areas, and the reservoir.

* Investigation of available construction materials such as earth, gravel, concrete aggregates.

* Any special features such as the possibility of earthquakes.

Sub-Surface explorations are necessary for collecting information relating to foundation conditions and to the natural materials available for construction. These should be planned only after the evaluation of all available geological and soil data. These are carried out to determine.

* Suitable and economic type of dam

* Foundation conditions and suitable construction materials

The principal purpose of subsurface exploration is to investigate the distribution, type and physical properties of sub surface materials by visual inspection and tests in the field or by securing samples of rock, soil and underground water for visual inspection and for testing in the laboratory.

The most satisfactory way to explore the underground is to dig test pits, trenches, shafts or tunnels so that one may get down there, examine the materials, take samples and test them. Due to high costs involved in these methods, it is necessary to rely on alternative methods, such as drilling and driving for obtaining undisturbed samples of overburden or burrow areas.

3.2 Feasibility

In the feasibility stage, the data are primarily qualitative. Detailed investigations are carried out in this stage to determine the technical feasibility of the project. Limited explorations are done to confirm the geological interpretation or to develop a new interpretation, if necessary. Pertinent conditions are developed and dimensions of structures are approximately determined. The field work is done jointly by a geologist and an engineer.

The objective of the feasibility stage of an investigation is to confirm or expand the work done in the reconnaissance stage in order to determine the essential plan and features and to prepare the scope and magnitude of the project and an adequate cost estimate. These data will be used for economic justification of the project. The purpose of feasibility investigation is to establish factual background for a design and estimate.

In feasibility stage of investigation a more specific project report will be prepared to obtain project authorization and approval to proceed with the construction.

After completion of the preliminary investigation one of the several sites is selected for final precise investigation which will help the engineer to have all pertinent data to proceed with the detailed designs of the structures and a more accurate cost estimates. Preliminary and final investigations often blends into one another. Final investigations are usually guided by the recommendations done at the preliminary level.

The purpose of the **final investigations** are,

* To determine the relative merits of two or more sites for the dam in question

* To determine the type of structure to be constructed

* To settle beyond doubt by sub-surface investigation, the nature of the foundations as affecting the

safety and cost of the dam and appurtenant works.

* To obtain all necessary information affecting the design of the dam and appurtenances.

3.3 Detailed Designs

In this stage the data are primarily qualitative and specific. These investigations are done to obtain data for preparation of designs and specifications. Sufficient explorations are done to establish conditions at all critical points. Engineering properties of soils and materials are determined. The work is done primarily by an engineer with geological assistance as required.

Investigations at this stage supplement detailed investigations carried out during feasibility stage to provide data from which designs for construction of various features of a project may be made and to finalize the specifications.

At this stage of investigation, the size and location of the structure under consideration are known and exploratory work is restricted to the area to be occupied by the structures. The investigations done at detailed design stage are for putting finishing touches to previous investigations.

The design of structures is well advanced at this stage and in comparing the design features with the available geological information, the design engineers may need additional bore holes for obtaining additional geological evidence to answer many critical questions posed at the feasibility design stage, such as,

* Extent of fault zone or presence of buried channels filled with highly pervious materials underneath the dam foundations

* Perviousness of the ground underneath the dam and the rim of the reservoir

* Capacity of the rock to carry some heavily stressed parts of the dam

* The depth of cut-off trench or extent of grouting operations

* The characteristics and depths of the subsurface geologic formations for preparation of detailed construction drawings and specifications.

During this stage detailed exploration of the borrow areas is also made for formulation of the plan for the exploitation of quarry and for taking decision on the selection of the construction plant.

3.4 Construction

Foundation and material investigations during the construction stage are primarily confirmative in character. These are carried out to clarify those conditions which had not been resolved during detailed design stage and to explore alternative proposals. Sometimes possibility of better source of material may also necessitate the additional investigations during construction stage. Work is performed under the direction of the construction engineer.

At this stage the geological investigations are performed to obtain information about specific areas of construction and of materials to resolve any design or constructional problems that is not possible to be solved at the detailed design stage. The problems are actually encountered and investigations are meant to resolve the specific problems that come up. A much clearer picture of the geological conditions becomes evident during the construction stage due to the exposure by the opening up of the foundations and construction of appurtenances like tunnels and adits. Final construction drawings are either prepared or further amended on such information.

3.5 Operation and Maintenance

Investigations are undertaken during maintenance and operation stage only when structure proves to be unsatisfactory or these become necessary for preparation of operation schedules. Such situations are handled according to specific problem involved.

Continuous monitoring of performance and behaviour of the reservoir appurtenances and the ground conditions, seismic activity etc. are essential to be able to predict and remedy any situation that could develop during the operation of the reservoir.

4. RESERVOIR INVESTIGATIONS

The geology of the reservoir area is also an important consideration in water resources development.

There may be possibilities of leakage through low saddles and narrow ledges along the peripheries of the reservoirs into adjoining valleys. Air photos are very useful in locating such area. In critical areas detailed investigation may be carried out. Sometimes observation holes are left on the outer slopes of reservoir rim and measurements are continued after the reservoir starts filling. Remedial grouting measures may be taken if necessary.

The reservoir rim should also be examined for possible landslides. Landslides into reservoir may produce great waves of water over the dam. It may be necessary to reinforce unstable reservoir slopes. Areas near intakes and spill way inlets should also specifically be seen for any possibility of slides.

5. FOUNDATIONS FOR DAMS

The safety of a dam is inseparable from the conditions of its foundation. Most of the failures of dams have been caused by inadequate foundations. Hence, for the Civil Engineer as well as the Engineering Geologist foundations constitute an area of high importance throughout the project. Therefore it is very important that this feature of the design should receive proper attention.

5.1 Foundation Conditions

Foundation conditions depend upon the geological character and thickness of the strata which are to carry the weight of the dam, their inclination, permeability, and relation to underlying strata, existing faults and fissures.

The different foundations commonly encountered are discussed below:

Solid rock foundations which have relatively high bearing capacity and resistance to erosion and percolation would make excellent foundations for most types of dams. In such cases, removal of any disintegrated rock together with the sealing of any seams and fractures may be the only treatment necessary.

Gravel foundations, if compacted are suitable for earth and rock-rock-fill dams and low concrete gravity dams. As gravel foundations are frequently subject to water percolation at high rates, Special precautions must be taken to provide water cut offs or seals.

Silt or fine sand foundations can be used for the support of low concrete gravity dams and earth-fill dams if properly designed, but they are not suitable for rock-fill dams. The main problems are settlement, piping, excessive percolation losses and erosion of the foundation at the downstream toe.

Clay foundations can be used for support of earth-fill dams but require special treatment. Since there may be considerable settlement of the dam if the clay is unconsolidated and the moisture content is high, clay foundations are seldom suitable for construction of low concrete gravity dams and unsuitable for rock-fill dams. Tests of the foundation material in its natural state are usually required to determine the consolidation characteristics of the material and its ability to support the superimposed loads.

Non Uniform foundations of rock and soft materials have to be used occasionally if reasonably uniform foundations cannot be found to construct a dam. Such unsatisfactory conditions can often be overcome by special design features.

4.3 Foundation Investigations

Foundation investigations are necessary to evaluate the foundation conditions in order to select a safe and satisfactory site for a dam or to adopt necessary provisions in design and construction to overcome the deficiencies in the foundations.

Foundation investigations should determine the following factors in case of **Concrete Dams**

* Depth of overburden, and weathered rock in the foundation area need to be removed to obtain a suitable rock foundation;

* Strength of rock mass in compression and shear and its behaviour with respect to settlement and sliding under operating conditions;

* Presence of seams, joint planes, fractures, solution channels, unhealed faults and extent of shattering, which may be the cause of excessive uplift or cause of leakage through foundations, and may also endanger stability of the dam.

Following factors should be determined in case of **Earth and Rock-fill Dams** :

* Depth of unconsolidated and other unsuitable materials;

* Shear strength of materials in foundations and their behaviour with respect of settlement under load and due to creation of reservoir;

* Presence of caverns, joints, solution channels in ledge rock which may cause excessive leakage;

* Presence of soluble materials in sufficient quantity in foundation material to be hazardous.

2.5 Foundation Problems

The history of dam disasters throughout the world reveals that problems often arise from undetected or inaccurately evaluated defects in the foundation. Therefore, engineering must be closely linked with geology in the design construction and continuing surveillance of a dam.

Foundation deficiencies may be related to the natural conditions of the foundation or to its treatment during construction. Differential settlement, sliding, high piezometric pressures and uncontrolled seepage are common evidences of foundation distress. Cracks in a dam, even relatively minor ones, may also indicate a foundation problem.

Concrete dams can withstand overtopping for at least a limited time without damage. A failure may be due to the inability of the foundation to bear impact of the overflow, rather than the resistance of the dam itself, which is likely to be more than adequate.

The safety of arch dams is highly dependent upon the strength of their abutments. Failure may stem from weakness in the rock resulting from saturation or deterioration, or excessive flood loading, or from abutment shearing under hydrostatic pressures. The erosion of foundation materials by overtopping may also cause failure of arch dams.

Most rocks have enough strength to resist the load imposed by a dam. But a rock mass may have bedding and foliation planes, joints,shears and faults. These can be natural channels for seepage water, which may carry away soluble materials and erode openings. The planes may also be deficient in shearing resistance and susceptible to weathering.

Foliation, has a tendency to break into thin sheets. The cleavage may allow water, air and other weathering agents to invade the rock mass. The foliation planes are generally conducive to slippage.

Practically all rock formations have joints, which are fractures along which there has not been any slipping. They form boundaries of individual blocks in the mass. In a dam foundation,joints may be of concern because the condition of the joint fillings is uncertain and the joints has the adverse potential of becoming a conduit for leakage under dam.

Among the more dangerous elements at a dam site are faults, fractures which have slipped. These are of particular concern because they may have caused physical alteration of the rock to the extent that the load-bearing capacity has been reduced. The fault zone may have been so snattered and crushed that it is unable to support the heavy loads of a reservoir. Its soft filling could be susceptible to squeezing or blowing out. It may also hinder the grouting of cracks. Faulting not only alters the conditions of the adjoining rock but also displaces foundation blocks so that rocks of contrasting characteristics are side by side. This may bring hard rock to bear on a soft rock, or a tight rock against one that might leak like a sieve.

The major faults at a dam site or in its environ must be examined to asses the probability of their future movement. Dams constructed on active faults may be stressed severely during such slippage. Disclosure of geologically recent movement at a proposed dam site is usually reason enough for abandonment of the site.

Resistance to erosion may depend more on bedding,foliation and jointing characteristics than on the strength of the rock. Closely spaced potential planes of breakage are vulnerable to disintegration under hydrostatic forces. Such weaknesses must be given special attention in areas where outlets and spillways will discharge.

Solubility of the rock underlying the reservoir must also be considered. Limestones and gypsum sometimes present problems when exposed to water under pressure. Limestones may have joints and bedding planes that provide paths of infiltration that facilitate rock solution. However.joint enlargement and cavern development in limestone usually are slow enough to be controlled during the life of a reservoir. The deterioration of gypsum may be rapid enough to create a hazard.

At some sites it is practically impossible to discover and to assess all geologic defects prior to construction. Moreover, there is little likelihood that drilling and sampling of the foundation materials will be so selectively accurate as to define the most critical zones completely. Only during construction and operation can there be assurance that the site has been fully tested. Not frequently, problems appear for the first time in the operational phase, despite conscientious efforts to detect them sooner.

5.4. Foundation Treatment

Most interesting, Most challenging and potentially, most dangerous part of a dam is its foundation. It is least known in advance and least visible after completion. Understanding and diagnosis of possible problems of the foundation are important in designing a safe and economical structure.

A Good foundation is of ample strength to withstand the weight of the structure and to prevent sliding. To be acceptable as a foundation for a dam, the rock must be sufficiently strong and bonded to remain intact under forces superimposed by the dam and reservoir, as well as by natural elements. It must also be impervious enough to prevent excessive seepage.

No matter how much foundation exploration and testing have been done, the designer should be concerned about the capabilities of foundation and should accommodate adequate foundation treatment to compensate the unknowns.

The proper treatment of large dam foundations is one of the major problems of modern construction. In order that the superimposed structure may function as designed, it is essential that the foundation support maximum loads applied to it. If the foundation rock in its natural state is inadequate, it may be possible to remedy the existing defects and improve the rock so that it will provide adequate support. The injection of grout into defective foundation is a practical method of bringing about such improvement.

In pressure grouting, mixtures of cement, water, or other materials such as bentonite, sand etc. are forced into confined and inaccessible spaces to consolidate the mass as a whole. The pressure grouting, is primarily used to eliminate seepage and to reduce uplift pressures in the foundation beneath the structure. It also increases the bearing strength of the foundation rock by filling any voids or fissures that may be present.

Seepage and uplift pressures beneath a concrete dam are generally reduced by grouting the foundation through deep closely spaced holes in one or more rows parallel to the axis of the structure. This is called "Curtain" grouting and is supplemented by a row of drainage holes drilled a short distance downstream and parallel to the grout curtain. These drainage holes are intended to relieve any hydrostatic pressures that may develop from seepage water passing through the grouted zone.

The foundation should be thoroughly explored before a grouting design is prepared for a dam site. Data obtained from this exploration are used to determine the type and the size of dam suitable for the site; properties of the foundation rock; locations of faults, fractures, seams and cavities; and preparation of suitable grouting plans.

Pressure grouting used in dam construction are of three general types such as,

* Low -pressure or blanket grouting
* Intermediate- pressure grouting
* High - Pressure or curtain grouting:

Low - Pressure or Blanket Grouting are used to seal and consolidate the foundation near the surface and is done first in a normal construction programme.

In concrete dam construction low-pressure blanket grouting is performed in the surface rock in the upstream third of the foundation area, or in the entire foundation area if necessary. Blanket grouting for earth dam construction, if needed, is usually confined to the near-surface foundation rock adjacent to the cut-off walls. It may be used under any type of dam to seal the near surface bed rock where zones of weakness occur.

Low pressure grout holes are usually drilled to depths varying from 3 - 15 metres. The axis of the hole is taken normal to the foundation surface except where the hole would be parallel to a seam. In such cases, the hole is drilled at an angle with the foundation so that it intersects the seam. Pressures used vary from 20-150 psi depending on size and type of the structure and on foundation conditions. In general, the pressures are maintained at the highest point that will not produce detrimental uplift of the foundation rock.

Intermediate - Pressure Grouting is used to effect a deeper seal in the foundation along the upstream edge of the structure.

In concrete dams, intermediate-pressure grout holes are drilled from the surface of the foundation rock just upstream from the edge of the structure or through pipes in the upstream fillet at the base of the dam. They range in depth from 15 m to 30 m depending on conditions encountered Pressures used depend on the weight of the concrete on the foundation at the time of grouting and on the geologic structure and strength of the foundation rock. Normally they range from 75 to 400 psi.

High -Pressure or Curtain Grouting is used to form the main cut-off curtain to reduce seepage beneath the structure.

It is used in both concrete and earth dams to form a cut-off curtain. In large concrete dams this grouting is generally done from a gallery adjacent to or slightly down stream from the axis and close to the foundation rock. Pipes are embedded to a maximum depth of 5 ft. into the concrete from the gallery during construction. The holes are drilled and then grouted under high-pressure through the pipes after concrete, has been placed to a reasonable height above the foundation. The depth of these holes vary with geologic and topographic conditions. Pressures from 100 to 500 psi are used in large dams. In a few cases pressures upto 1000 psi have been used.

These pressures are localised by packers in the holes. To avoid heaving or uplifting in the foundation, high pressures should be used with caution and never in a haphazard manner.

Grout Mixes most widely used is a mixture of Portland cement and water, proportioned by volume. The water cement ratio depends on the tightness of the foundation rock. If the rock contains large voids which accept grout readily, the water-cement ratio may be 1:1. For tight seams or cracks in the rock, ratios of 20:1 have been used. Grout mixes usually range between 1:1 and 10:1. Cement grout mix when properly placed has relatively high compressive strength. It is used to consolidate foundation rock, to form cut-off curtains under dams, and to grout contraction joints in large mass-concrete structures.

Clay cement mixtures may be used as grout when a sealing filler is required in parts of the foundation that do not require strengthening. It is used to grout cavities and to form blankets or sealing mats within porous rock and alluvium. Clay grout mixture should have minimum adhesiveness and be free from organic or foreign matter. Minimum shrinkage of clay-cement grout is obtained with three parts of clay to one part of cement by volume. Sufficient water must be added to this mixture to make it suitable for pumping.

Bentonite is a soft, moisture absorbing, colloidal clay having properties which, under special conditions, make it useful as grout. It will absorb three or more times its own weight of water and, in doing so, will increase in volume seven or more times its dry bulk volume. It is easily pumped when mixed with water and is fine grained. Bentonite may be mixed with sand or cement to grout off large flows of water which cannot be grouted effectively with cement grout.

Asphalt Grouting has been used successfully where there is large subsurface flows of water which is difficult to stop by cement or clay grout. It has been particularly effective in sealing water courses in under ground rock channels. It has also been used to plug leaks in coffer dams and in foundation rock. This is expensive and should be used only to grout rock that cannot be sealed by other methods.

Asphalt with a melting point between 165° and 175°F has been found suitable for grouting. It is heated from 400°F to 500°F before it enters the pump. The asphalt should be free from trash, gravel or other debris.

Treatment of Faults and Seams

Narrow seams and faults can be washed and grouted while wider seams filled with soft material can be excavated and refilled with concrete. When such defective material lies in a nearly horizontal plane, below the surface of the completed excavation, it may be economical to reach it by a vertical shaft or large - size drill holes. Clean out the seam in drifts and fill with concrete rather than excavate the firm rock above it. Cavernous rock and solution channels may also be treated in this manner.

Vertical transverse faults of considerable size can be cleared and filled with concrete for a depth only sufficient to provide an arch to span the opening. Care should be taken that excavating and grouting extend far enough to obtain a tight cutoff at the upstream side.

Differential Settlement at irregular rock surfaces under the weight of the dam & reservoir is a common problem in the dam foundation. The resultant cracking of the embankment is one of the most threatening condition to be encountered. Preparation of rock foundations therefore should include shaping of projections and over hangs by removal and filling with concrete or shot crete.

Uncemented Shale

Rock such as uncemented shale tends to disintegrate when exposed. The final trimming should not be done until just before the concrete is to be placed. Otherwise, the rock might dry out and when again saturated by the water from the concrete might form a layer of mud between the foundation and the dam, offering no bond and little resistance to sliding.

If the disintegration of uncemented shale take place quite rapidly, the final foundation, as soon as uncovered, should be coated immediately with a bituminous or asphaltic water-proofing material. This procedure is very important to soft shale and adds to the strength of the bond between the shales and the concrete.

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DISPERSIVE CHARACTERISTICS OF SOILS USED IN THE CONSTRUCTION OF EARTH DAMS & EMBANKMENTS

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1.0 INTRODUCTION

Problems associated with dispersive soils have been reported in many parts of the world such as United States, Australia, Greece, France, India, Latin America, South Africa & Thailand. Two major earth dams namely Teton & Baldwin Hills in USA, failed by breaching just after the ponding of the reservoir, during the years 1976 and 1963 respectively. This was not understood for a long time. Breaching of Teton dam had been a very significant event for the Geotechnical Engineers concerned with the design and construction of earth dams, as no dam of such a height had previously failed. The failed section was completely washed away in the disaster.

It had been thought that Teton dam failure mechanism would never be identified as the whole failed section was washed away during the disaster. During excavations of the embankment left, many wet seams have been found. But investigators couldn't arrive at any conclusion because there was not enough evidence. In Baldwin Hill dam, it has been noticed that a large amount of water was seeping through underneath the foundation before failure. When the geology of sites of failed dams were analysed, Baldwin Hill was found to be located in soils of marine origin.

It was noticed that in almost all the cases, dispersive piping has occurred during ponding of the reservoir. In very few cases only, failure occurred after the reservoir level was raised. The failures usually started with an initial leak which gradually eroded to form a tunnel. In some cases the entire reservoir volume was discharged through a channel. In some cases failure was discovered only after the reservoir had been emptied. In cases of erosion of embankment by rainfall, tunnels and deep gullies were important features.

After several investigations, it was concluded that the chemistry of soil used in the construction of dams that failed is much different from the ordinary erosion resistant clays, and it became necessary to know the mineralogy of them. Such soils were classified as dispersive soils, where percentage of sodium in pore water is much higher than that in the ordinary erosion resistant soils, which have calcium and magnesium in high percentages. It was found that these types of soils occur mostly in areas where tropical humid climatic conditions prevail. Later researches tend to show that even with presence of high sodium, dispersion does not occur when there is no montmorillonite clay mineral present. Thus dispersiveness seems to be more a result of sodium adhering to the montmorillonite present than the presence of sodium.

Presence of sodium cations could be from several sources such as clays which are derived from marine sediments in the geologic cycle, or as a result of the geologic weathering.

The purpose of this paper is (1) to describe the factors influencing piping failure in dispersive soils. (2) to describe the identification tests & quantitative tests used & (3) to explain methods used for stabilising such soils. Also evaluating dispersivity by different criteria is reviewed.

2.0 FACTORS AFFECTING THE DISPERSIVE CHARACTERISTICS OF CLAY

It has been identified by several writers that the factors affecting dispersive characteristics are (1) Sodium absorption ratio of soil, (2) Total dissolved salts in the soil, (3) Total dissolved salts & Sodium absorption ratio of the percolating water and (4) Type & amount of clay mineral present in the soil.

2.1 Sodium absorption ratio

This is a measure of relative amount of sodium in the soil where Na, Ca, and Mg are dissolved salts in milliequivalents per litre of ǂsaturation extract and calculated by the formulae;

$$SAR = \frac{Na^{+}}{\sqrt{0.5(Ca^{++} + Mg^{++})}}$$

The range of SAR is from 0 to infinity. This is closely related to the exchangeable sodium percentage (ESP). Total cation exchange capacity is measured in terms of milliequivalents per hundred grams of soils. Relative amounts of the four basic cations on the "exchange complex" is measured. Then ESP is expressed by,

$$ESP = \frac{Na}{CEC}(100)$$

From the case studies of various writers it is found that there is strong correlation between sodium absorption ratio and the exchangeable sodium percentage in the saturation extract.

As measurement of ESP is difficult, SAR test is usually performed.

Many case studies have shown that high sodium content clays erode very rapidly. Higher this percentage, greater the susceptibility to erosion. High values of SAR reduces the soil permeability considerably, which is explained later in this paper.

Agricultural scientists were aware of this fact long ago. Soils with ESP > 15 and low total salts in the pore water have caused trouble in agriculture because the individual clay particles become suspended in raindrops falling on the ground surface and they are carried down into the soil with percolating water where they accumulate and form a dense layer of low permeability, which makes the ground difficult to irrigate and drain.

2.2 Total dissolved salts in the pore fluid of soil samples

Total dissolved salts in saturation extract = Ca + Mg + Na + K is measured in milliequivalent per litre.

It has been proved that higher the salt content in the saturation extract, the more resistant is the clay to erosion. That is clear because for a soil with porefluid TDS 20-30 meq to be dispersive, sodium percentage should be high as 50. Generally samples collected from the failure portions had less than 15 meq per litre. TDS > 40 meq/l are flocculated in their natural state (saline & saline alkalai) but in laboratory tests they become deflocculated due to the dilution of salts.

2.3 Total dissolved salts and sodium absorption ratio of the percolating (eroding) water.

Case studies in Australia and Israel have shown that likelihood of clay dams failure by dispersive piping was much higher when the reservoir water was pure. In many of the failed dams, reservoir water had total dissolved salts less than 5 meq/litre. At Oklahoma where many dams failed, the salt content in the reservoir was very low ranging between 0.3 and 2.5 meq/litre. This was noticed in the failed dams in Australia. It could be said that lower the total dissolved salts in the percolating water, greater is the susceptibility to erosion. In other words, amount of osmotic swell induced in a clay - water electrolyte system depends on the difference between concentration of pore fluid of dam structure and percolating water.

2.4 Clay Mineralogy

Study of clay mineralogy is important to understand the failure mechanism of dispersive soils. Clay minerals are formed primarily from the chemical weathering of certain rock-forming minerals. However their nature depends on the mineral composition of the parent rock and upon the physico-chemical environment in which the weathering takes place. All clay minerals are very small, colloidal sized crystals

ǂsee Appendix 1

(diameter less than 1 µm) and can be seen only with an electron microscope. Techniques such as X - ray diffraction, differential thermal analysis, dye absorption and chemical analysis are also available for determining mineralogical composition of soils. However these tests could not be carried out in normal soil testing laboratories due to the lack of specialised apparatus and expert interpretation. Atterberg limits give some clue regarding the amount & type of clay minerals present in soils.

All clays have two fundamental crystal sheets, tetrahedral or silica and the octahedral or alumina. The way sheets are stacked, together with different bonding and different metallic ions in the crystal lattice cause different clay mineral. Kaolin, montmorillonite and gibbsite are the main clay minerals found in Sri Lanka.

From the current study, it seems that most of the dispersive soils taken from the damaged dams had substantial quantities of montmorillonite. Montmorillonite, sometimes called as smectite, is a three layer mineral having a single octahedral sheet sandwiched between two tetrahedral sheets to give a 2 to 1 lattice structure. Because the bonding by Vanderwaals' forces between the tops of the silica sheets is weak and there is a net negative charge deficiency in the octahedral sheet, exchangeable ions can enter and separate the layers. Calcium and magnesium are the predominant exchangeable ions. The depositional environment as well as subsequent weathering and leaching will govern what ions are present in a particular soil deposit. It has been pointed out by Tourtelot (N.W.Herath (1993)) that the setting for the formation of montmorillonite is extreme disintegration, strong hydration and restricted leaching so that Mg, Ca, K and Na cations may accumulate in the system. He further indicated that such conditions are favourable in the regions of tropical environment, with highly seasonal moderate rainfall, particularly where evaporation exceeds precipitation. This suggests that marine clays will be predominantly sodium and magnesium since these are the most common cations in the sea water.

The atomic structure and schematic diagram of montmorillonite clay mineral is given in Fig 1 & 2. This clay mineral is plotted high above the A - line and close to the U - line in the plasticity chart and the Atterberg Limits test is used as a simple approach in identifying the clay mineral.

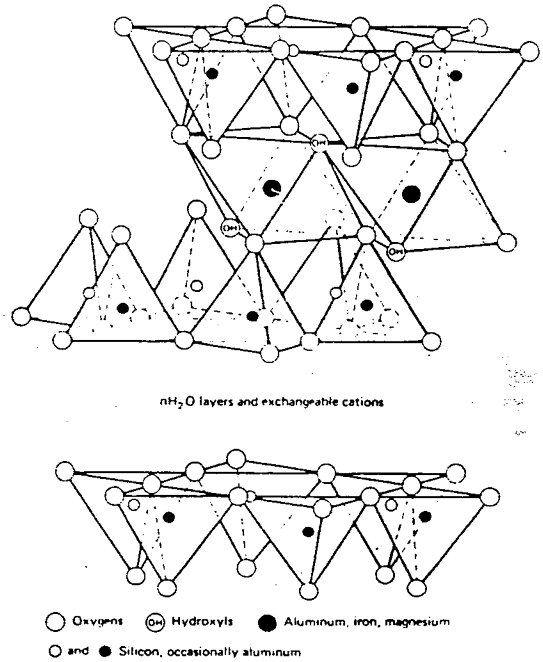


Fig 1 - Structure of Montmorillonite clay mineral

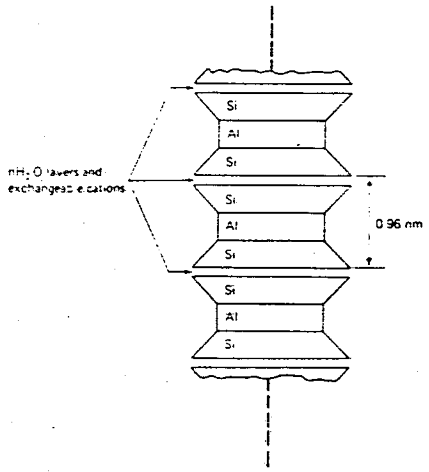


Fig .2 - Schematic Diagram

In Sri Lanka clay mineral provinces closely follow the climatic zones of the Island as shown in Fig 3. Dry zone has kaolinite - montmorillonite clays. Sedimentary limestone deposits of miocene age are best developed in the Jaffna Peninsula. These deposits extend up to Puttalam.

Intermediate zone climatic conditions played a major role in the nature of clay mineral development in Sri Lanka. Progressive development of montmorillonite from wet to dry zone areas is one of the characteristic features noted, which is in agreement with Tourtelot's opinion.

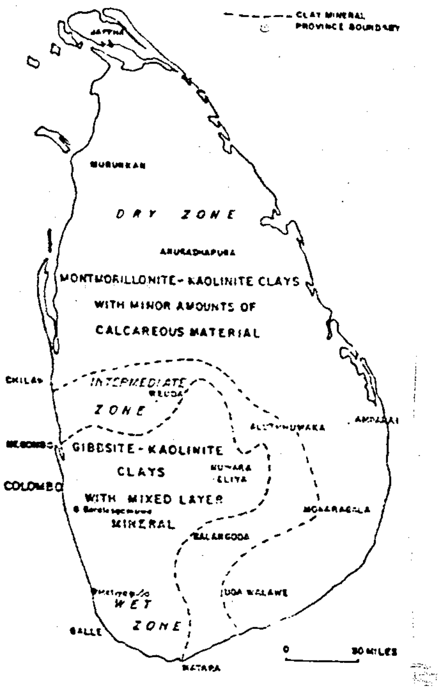


Fig. 3 - Clay Mineral Provinces - Sri Lanka

3.0 MECHANISM OF PIPING IN DISPERSIVE SOILS

Small leaks through and under earth dams are fairly common and statistically only a small percentage of such leaks threatens or causes failure.

Engineers are aware that many clay dams had leaks through them for years without piping. In non dispersive soils, erosion could not occur when velocity of water is below a certain value called as threshold velocity and there exists a value called critical shear stress. But in dispersive soils, the colloidal clay particles go into suspension even in still water.

This is because dispersive soils contains a clay fraction which has a high potential to be in dispersed state when the soil mass interacts with water. Sodium is considered to be the main factor for dispersivity but not the sole factor.

Sodium has an ionic charge of plus one. Clay particles have negative charges at their surfaces. For the same negative charge on a clay particle, the number of sodium ions around will be double that of divalent ions like calcium or magnesium. Then repulsive forces between clay particles surrounded by sodium ions are higher than when they are surrounded by divalent ions.

This could be explained by the theory that when there is double layer with two similar charge surfaces, then the repulsive action is great to prevent closer approach of particles for Vanderwaal's attractive forces to be operative and therefore clay suspension stays in the deflocculated state.

Furthermore, because of its single valence, the Coulombic attraction of a sodium ion to the charged particle surface is less than that for polyvalent ions. Hence sodium ions stays a little away from clay particle which increases the double layer thickness.

The flocculating power decreases in the order of lytropic series $Fe^{++} > Al^{+++} > Ca^{++} > Mg^{++} > NH_4^{+} > K^{+} > Na^{+} > Li^{+}$. Since Na has a lower valence and larger hydrated radius than Ca or Mg, sodium clays are more likely to exhibit a higher degree of dispersion. As a result, swelling takes place and permeability is decreased in the presence of high sodium. Swelling results in blocking or partial blocking of the larger conducting pores (Note: permeability is proportional to fourth power of the radius). Dispersion can be regarded as occurring when the charged plates, which are moving apart in the process of swelling, have reached such a distance of separation that attractive forces are no longer strong enough to oppose the repulsive forces.

When a concentrated leak starts through a clay embankment with high ESP, either of two actions can occur (The initial narrow crack could be caused by drying and shrinkage of plastic material, differential settlement, hydraulic fracturing or merely poor compaction):-

(i) if the velocity of flow is sufficiently low, the clay surrounding the flow channels swells and progressively seals off the leak.

(ii) if the initial velocity is sufficiently high, the dispersed clay particles are carried away, enlarging the flow channel at a faster rate than it is closed by swelling, leading to progressive piping failure.

It should be pointed out that the nature of piping failure in dispersive soil is quiet different from that which occurs in cohesionless soils which is commonly described in soil mechanics literature. The erosion just starts at the discharge end of the leak, causing a local enlarged tunnel-like leakage channel to gradually extend upstream by progressive erosion until it reaches the water source supplying the leak, at which time a rapid catastrophic failure may result.

Case studies of dams failed by dispersive action show that the leak was traveling through concentrated leakage channel from the moment of its inception. The erosion of the wall channel also took place at the same time.

Statistically it has been shown that piping failures are more common in clay dams than in dams constructed with low cohesion soils .

4.0 LABORATORY IDENTIFICATION TESTS

Four laboratory tests for identifying dispersive clays have been used by the researchers viz, (1) Sodium Absorption Ratio Test, (2) SCS Laboratory Dispersion Test, (3) Crumb Test & (4) Pinhole Test

4.1 Sodium Absorption Ratio Test

This test has been used by the agricultural scientists for long. A soil is mixed with water and a liquid limit consistency sample is prepared and left overnight in an air tight container. Pore water is extracted by vacuum using a filter. Saturation extract is tested for Na^{+}, Ca^{++} & Mg^{++} concentrations in milliequivalent per litre by an atomic absorption spectrophotometer which is equipped with a digital read out unit.

$$SAR = \frac{Na^{+}}{\sqrt{0.5\ (Ca^{++} + Mg^{++})}}$$

Total dissolved salts too could be calculated from above test, provided potassium content is also measured by following expression; TDS = Ca + Mg + Na + K, measured in milliequivalents per litre.

4.2 SCS Laboratory Dispersion Test

This test, also known as double hydrometer test, has been widely used by the Soil Conservation Service which builds many small irrigation, flood control structures all over the United States. In this test, sample is tested for finer particle size distribution by the standard hydrometer test in which a soil water suspension is prepared by breaking down the soil with a chemical dispersant and strong agitation. Another test is made on the soil water suspension of same soil prepared without dispersing agent and strong agitation. Sodium hexametaphosphate is used as the dispersing agent. Percent dispersion is calculated by the expression below:-

$$= \frac{\text{\% finer than 0.005 mm without dispersing agent \& strong agitation}}{\text{\% finer than 0.005 mm by standard hydrometer test}} \times 100$$

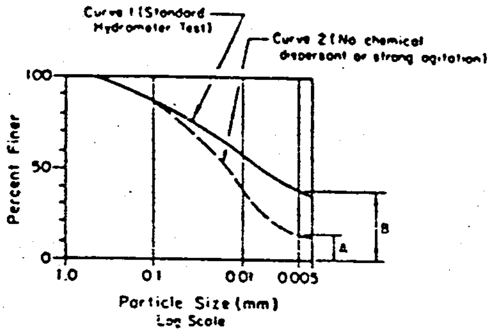


Fig.4 - SCS dispersion test

Soil Conservation Service concluded that soils with dispersion ratio 35 - 50 % could be considered as moderately dispersive and 50 - 75 as highly dispersive. For ordinarily erosion resistant clays this value is between 10 and 35 %.

This gives an idea of how much the soil is dispersed in naturally existing state.

4.3 Crumb Test

A crumb of soil is immersed in a small beaker of water (150 ml) or 0.001 Normal NaOH solution and the tendency for the clay particles to go into colloidal suspension is observed after 5 - 10 minutes.

Following interpretation is used for evaluation of results:-

Grade 1. No reaction: Crumb may slake and run out on bottom of the beaker in flat pile but no sign of cloudy water caused by colloidal suspension.

Grade 2. Slight reaction: Bare hint of cloud in water at the surface of crumb.

Grade 3. Moderate Reaction : Easily recognisable cloud of colloids in suspension. Usually spreading out in thin streaks on bottom of beaker.

Grade 4. Strong reaction: Colloidal cloud covers nearly whole bottom of beaker, usually in a very thin skin. In extreme cases all the water in the beaker becomes cloudy.

Crumb test is a very good indicator, but only in one direction. If crump test gives a soil as dispersive, that will be definitely dispersive. But not vice versa.

4.4 Pinhole test

This test was first developed by Sherard in 1970 and later modified. In this test distilled water is percolated through a 1.0 mm diameter hole in a compacted specimen of specified dimension at different heads. Fig.5 shows a modified version of the pinhole apparatus which is in use and a section through test specimen. The rate of flow is measured & colour of water is noticed. For dispersive soils, water becomes coloured and the hole is rapidly eroded. Compaction of the sample is done close to the plastic limit or optimum water content by Harvard miniature compaction hammer. This is a direct method of identifying dispersive characteristics, because it simulates the actual field conditions.

Result is evaluated from the appearance of the water, rate of flow, and final size of the hole in the specimen. Fig. 7 & 8 gives typical test data for a dispersive and nondispersive soil respectively and Table 1 the criteria for classifying the soil into one of six categories. Rate of flow for different head has been obtained from pipe flow computation for laminar flow for a diameter of 1 mm, the size of punched hole.

Results of this tests have shown that a non - dispersive clay could withstand a small crack without erosion when water is flowing at 10 fps (3.05 m/s). In a dispersive clay erosion could be seen even at 1 fps (0.305 m/s), corresponding to a head of 2".

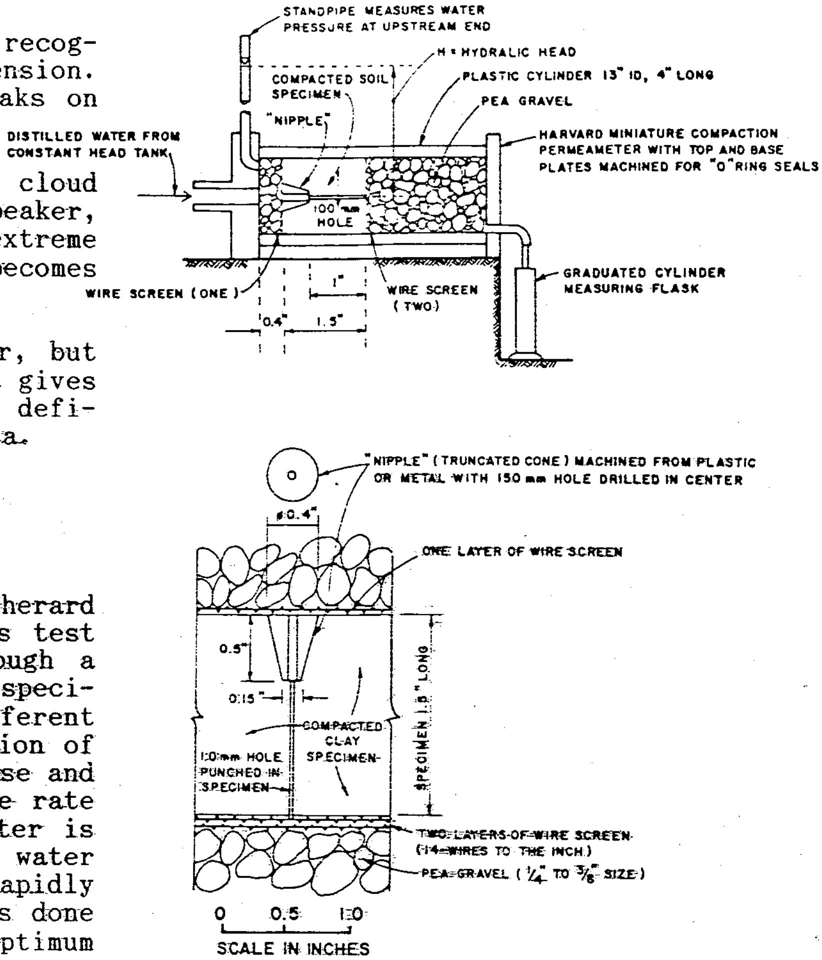


Fig.5 - Pinhole Test Apparatus and Section through Pinhole Test Specimen

Variation in the results for the same sample has been noted when the sample is tested at natural moisture content and after air drying, where chemical test indicates the sample to be dispersive. For consistent results, it is preferable to cure the sample before testing by pinhole method; viz, compacted sample is pushed out of the mould and cured in a polythene bag for a day or more. Substantial changes in the clay particle interaction force takes place during curing.

TABLE 1

Criteria for Evaluation of Results by Pinhole Test

Classification	Head in inches	Test time for given head minutes	Visual final flow through specimen ml/sec	Colour of flow at end of test (cloudy or colour)	Hole size after test
D1	2	5	> 1.5	Very distinct	2x
D2	2	10	> 1.0	Distinct to slight	2x
ND4	2	10	< 0.8	Slight but easily visible	1.5x
ND3	7-15	5	> 2.5	Slight but easily visible	2x
ND2	40	5	> 3.5	Clear or barely visible	2x
ND1	40	5	< 5.0	Crystal clear	No Erosion

PIN HOLE TEST DATA

Pin Hole Test No. Example
Sample No.
Compaction Characteristics good
Water Content near optimum
Distilled water added: ✓ (Yes) or ___ (No)
Curing time: none

Date:
Page:
Specimen after test: Final hole about 2.5 mm
Flow started on 1st trial.

Clock Time	Head	Flow Rate ml	Flow Rate sec	Color from Side: Dark	Slight to Medium	Barely Visible	Completely Clear	Completely Clear from Top	Particles Falling: None	Few	Heavy	Remarks
3:05	2"	10	25	✓					✓			highly colloidal
		10	10	✓					✓			flow
		25	18	✓					✓			
		25	18	✓						✓		
		25	17	✓						✓		
		25	16	✓						✓		
		25	15	✓						✓		
3:[illegible]		25	15	✓						✓		Stop test-
												Obvious Failure
												(D1)

Fig.7 - Typical Data for a Dispersive clay

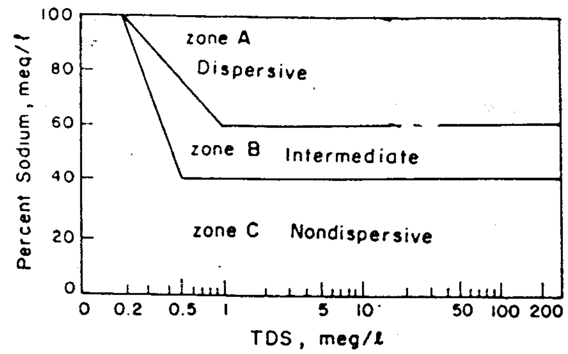
PIN HOLE TEST DATA

Pin Hole Test No. Example
Sample No.
Compaction Characteristics Soft sample
Water Content 19.3%
Distilled water added: ___ (Yes) or ✓ (No)
None.
Curing time: Compacted at natural water content

Date:
Page:
Specimen after test: no erosion of hole
Flow started on 1st trial.

Clock Time	Head	Flow Rate ml	Flow Rate sec	Color from Side: Dark	Slight to Medium	Barely Visible	Completely Clear	Completely Clear from Top	Particles Falling: None	Few	Heavy	Remarks
9:33	2"	10	22				✓	✓	✓			Sparkling Clear
		10	20					✓	✓			
		10	19					✓	✓			
9:38		10	19					✓	✓			
9:38	7"	25	20					✓	✓			
		25	20					✓	✓			
9:42		25	20					✓	✓			
9:43		25	20					✓	✓			
9:43	15"	50	27					✓	✓			
		50	27					✓	✓			
9:45		50	26					✓	✓			
9:47		50	27					✓	✓			
		50	26.5					✓	✓			
9:48	40"	50	15					✓	✓			
		50	15					✓	✓			
9:50		50	16					✓	✓			
		50	15.5					✓	✓			
9:52		50	15					✓	✓			
		50	16					✓	✓			
9:53		50	16					✓	✓			End Test (ND1)

Fig.8 - Typical Data for a Nondispersive clay



$$\text{Percent Sodium (meq}/\ell) = \frac{Na(100)}{Ca+Mg+Na+K}$$

$$TDS = Ca+Mg+Na+K$$

= Total Dissolved Salts

Fig.6 - Relationship between dispersibility and total dissolved pore-water salts based on pinhole tests.

Sometimes slightly coloured water flows at the beginning of test but becomes clear in a few seconds. This may be due to loosely held fragments being washed away and should not be mistaken for a dispersive clay.

Strong correlation was found between pinhole test results and SAR of pore fluid of the sample and Fig. 6 has been established with strong evidence.

It is observed in all the documented findings that the above summarised criteria fit well.

4.5 Test Results

Test results on the 25 samples obtained from earth dams of Sri Lanka for SCS dispersion test, Pinhole test & Crump test carried out at the Civil Engineering Division Laboratory of Open University are tabulated in Table 2.

TABLE 2

No	District	Name of tank /Borrow area	Degree of dispersion	Pinhole test	Crumb test
1	Ampara		69.0	D_2	Grade 1
2	Colombo	Kelaniya*		ND_1	Grade 2
3	Galle	Kendagas Mandiya anicut	96.0	ND_2	Grade 1
4		Kiripada wewa anicut			Grade 1
5		Pahala Liyedda anicut			[illegible]
6	Hambantota	Walawe		ND[illegible]	Grade 2
7		Lunugamwehara 1		ND_1	Grade 1
8		Lunugamwehara 2			Grade 1
9	Kaltota	Thanjan Tenna Tank		ND_1	Grade 2
10	Kantalai	Wan Ela	59.3	ND_3	Grade 2
11	Kurunagala	Iridiyagama (01)	6.5	ND_4	
12		Iridiyagama (02)	49.2	ND_4	
13		Iridiyagama (03)	6.9	ND_4	
14		Hakwatuyana oya	50.0		Grade 2
15	Matara	Nilwala ganga (01)*			Grade 1
16		Nilwala ganga (02)			
17	Polanaruwa	Kaudulla (01)			
18		Kaudulla (02)	78.3	ND_1	Grade 2
19		Kaudulla (03)			
20		Kaudulla (04)		ND_1	Grade 1
21	Puttalam	Nedunkulam	68.8	D_2	Grade 2
22		Inginimitiya	94.6	D_2	Grade 3
23		Kachchimaduwa		ND_1	Grade 1
24	Vavuniya	Pavatkulam		ND_2	Grade 1

*** It was unable to do hydrometer test without dispersing agent as the soil in the suspension settled very quickly.**

5.0 QUANTITATIVE TESTS

5.1 Rotating Cylinder Test

Other than these four tests, an apparatus was built at the Soil Mechanics Laboratories of University of California at Davis by Arulanandan et al (1975), with minor modifications to that originally used by Masch, Epsey and Moore, to measure the resistance of soils to surface erosion by water. In this test two concentric cylinders are separated by an annular space of 0.5" as shown in Fig.9. A cylinder of cohesive soil, 3" in diameter and 3.2", long is mounted concentrically inside the large transparent cylinder that could be rotated at speeds upto 1,500 rpm. To transmit shear from the outer rotating cylinder to the surface of the soil sample, the annular space between the sample and rotating cylinder is filled with the eroding fluid. As the outer cylinder is rotated with the inner cylinder (soil sample) held stationary, rotation is imparted to the fluid. This movement of the fluid, in turn, transmits a shear to the surface of the inner soil. The soil sample is mounted on a bearing.

Flow in between the two concentric cylinder is parallel and has only tangential component. When the inner cylinder radius at rest is R_1, outer cylinder radius is R_2 and rotating angular velocity is w, it has been shown that Torque, T transmitted by the rotating cylinder to the fluid is equal to ,

$$T = 4\mu h\pi \frac{(R_1 R_2)^2}{R_2^2 - R_1^2} w$$

Torque developed on the inner cylinder is equal to,

$$T = 2(R_1)^2 \pi h \tau$$

By equating T in both equations,

$$\tau = 2\mu \frac{R_2^2}{(R_2)^2 - (R_1)^2} w$$

where h is the height of the cylinder; μ = the viscosity; τ = shear stress on the sample .

Sample is placed in the erosion apparatus and the amount of material eroded was determined from the differences in weight of the sample before and after applying the shear stress. This is repeated for same sample for varying rotating speeds and the erosion rate vs shear stress graph is obtained. Arulanandan et al (1975) repeated this series of tests for same sample after increasing the SAR value by soaking in solutions of sodium salt of different concentrations and a set of test results is given in Figs. 10 (a) , (b) & (c). The intercept on the applied shear stress is called the critical shear stress. It was noted that for soils with high SAR these values were closer to zero. By combining the results from Fig 10 (a), (b) & (c) the variation of critical shear stress with SAR was obtained as in Fig.11.

Zero critical shear stress was considered a better basis for the definition of dispersive clay by him.

Arulanandan et al (1975)'s study on dispersive soil by rotating cylinder apparatus is in good agreement with the boundary for dispersive and non dispersive states given by upper curve of Fig.6 previously established by pinhole tests & chemical rests.

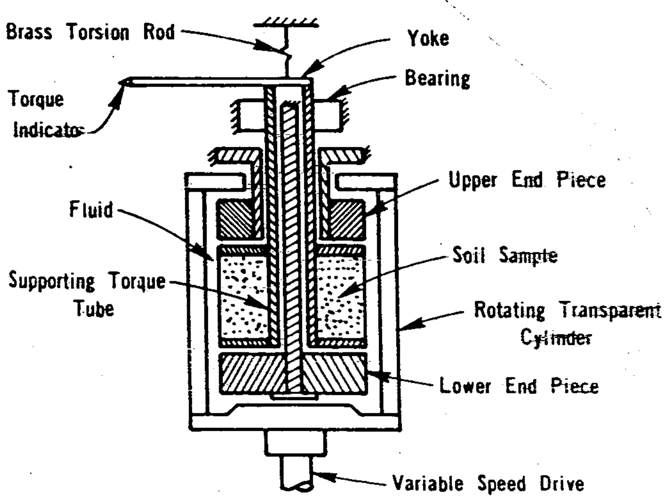


Fig.9 - Cross - Sectional View of Rotating Cylinder Test Apparatus

However after performing the erosion test, at Davis, Heinzen and Arulanandan (1977) claim that pinhole test is relatively insensitive since a head of only 1/2 cm is required to produce a shear stress in the order of 10-15 dynes/cm^2.

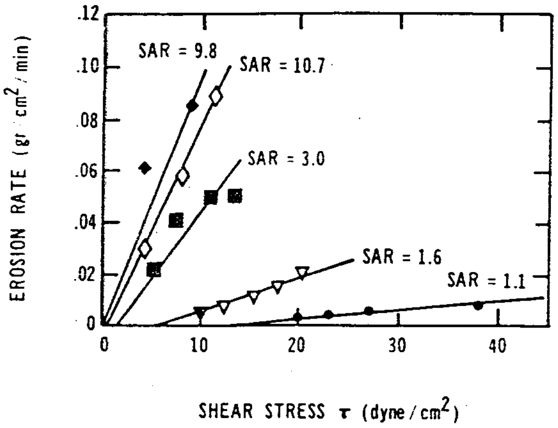


Fig 10 (a) - 0.005 N Concentration

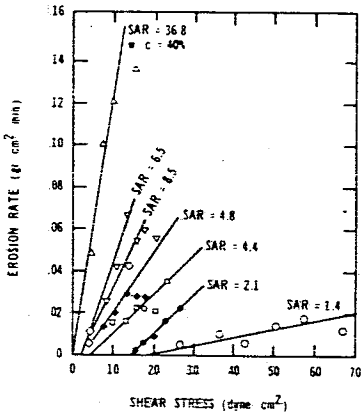


Fig 10 (b) - 0.01 N Concentration

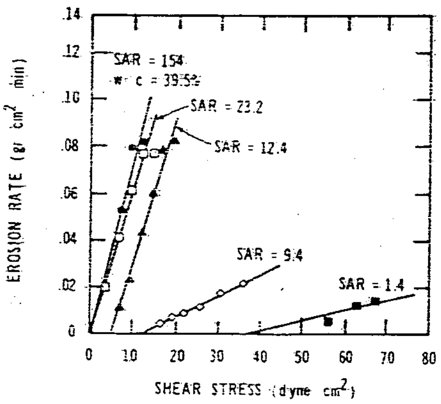


Fig 10 (c) - 0.1 N Concentration

Fig 10- Relationship between Erosion and Shear Stress for Different SAR (Yolo Loam samples) (Reproduced from Arulanandan et al (1975).

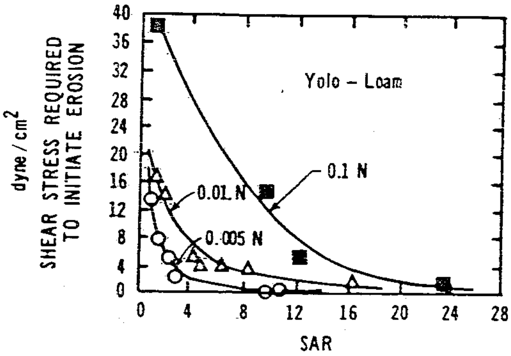


Fig.11 -Relationship Between Critical Shear Stress and SAR for different pore water electrolyte (Reproduced from Arulanandan et al (1975).

Heinzen and Arulanandan (1977) analysed the results of various erosion studies at University of California and concluded that $\tau_c = 0$ is the best criteria for dispersive soils. A comparison basis was used to evaluate the dispersion ratio. However this ratio obtained by this basis for limited sample was greater than that by double hydrometer test, proposed by Soil Conservation Service & Sherard. This discrepancy arises from the differences in the criterion used. The dispersibility as per τ_c value obtained by the rotating cylinder and flume is considered more precise than by the earlier subjective definition of dispersive clays.

5.2 Recirculating Flume

Another quantitative method is measuring the resistance to surface erosion to soil by a recirculating flume. This has been used by Kandiah et al and Heinzen. (Arulanandan et al (1977)).

A schematic diagram of modified version of this flume and sample containers, used for surface erosion test by Shaikh et al (1988) is given in Fig. 12. The flume has three controls for varying the velocity of fluid flow; a discharge control valve, slope adjustment, and a tailgate position. The smooth clear plexiglass flume bed is modified so that surface of the sample was flush with the bottom of the flume. Water was recirculated in the system. The depth of flow ranged from 0.8 - 2.1 cm. The flow rate was measured by precalibrated Venturi meter, and uniformity of the flow was manipulated by an upstream control. The velocity profiles along the flume at sections shown in Fig. 12 were measured using a pitot tube. The tractive shear stress τ exerted on the flume was estimated by measuring velocity profiles and using the Prandtl-Von Karman equation for smooth channels. Erosion rate of samples were calculated at the end of each test for 3 different slopes of bed.

Samples were chemically treated, compacted and pushed into the containers at high pressure. Surfaces of the samples were trimmed to flush with the top edges of container and placed in the flume. Wet weight of the sample was measured before and after circulating water.

However, Shaikh et al claims that the piping failure by internal erosion could not be correctly predicted by τ_c values obtained by above surface erosion tests. Further he claims in case of unsaturated clays erosion rate is further complicated by a process called slaking.

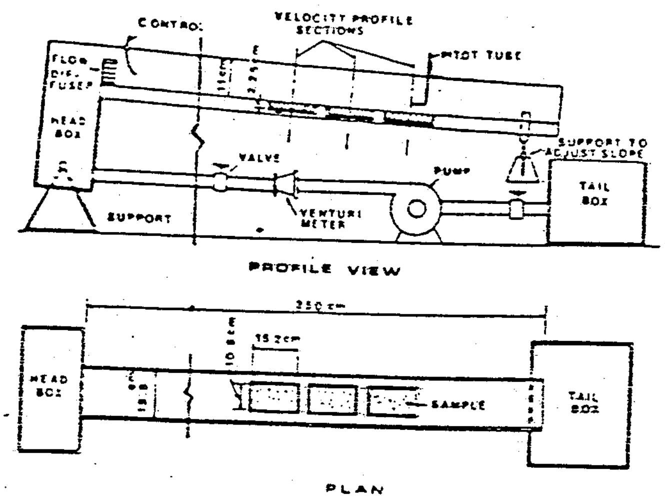


Fig.12 Schematic Diagram of Recirculating Flume with Sample Containers

5.3 Free Swell Test

Free swell tests show that constant water uptake is a function of the type and amount of clay mineral. This also could be used as a basis for evaluation of dispersive clay.

This test is made on soil pads of a few millimeters thick which are saturated with pore fluid of same composition which existed in the soil and then allowed to swell on sponges soaked with distilled water. The pads are weighed periodically until a constant weight has been reached. After that soil pad is dried and weighed.

6.0 EXCEPTIONS

Two homogeneous dams of clay failed by piping on the ponding of reservoir; the Washington County Dam in Illinois and the Stock Creek Dam in California failed in 1962 and 1950 respectively. Both dams failed by breaching with failure details very similar to Oklahoma dams.

But the chemical test plotted were in the non-dispersive region (zone C of Fig.6) and the results of the dispersion tests were low. Though many reasons were suggested, I believe the cause has not yet been identified so far.

7.0 ATTERBERG LIMITS, COMPACTION WATER CONTENT

It was noted that Atterberg limits give no clue whether a soil is dispersive or not. However it has been noticed the ML - MH or MH soils are mostly non-dispersive, and dispersive clays are predominantly CL, i.e. silty clay of low to medium plasticity, with some silty sand, sandy silt and clayey silt. There have been cases where tough CH clays are found to be dispersive.

Further it has been noticed that test results are generally not affected by moderate differences in compaction water content but in intermediate category (ND_4 and ND_3) can have an important effect due to the reason explained in section 4.4.

8.0 METHODS OF STABILISATION

Lime could be effectively used to stabilise dispersive soils. Addition of hydrated lime tends to increase the total concentration of calcium cations and reduce the sodium percentage, hence controlling the dispersivity. The general order of replaceability of the cations is given by the lytropic series $Fe^{++} > Al^{+++} > Ca^{++} > Mg^{++} > NH_4^+ > K^+ > Na^+ > Li^+$. Any cation in this series will tend to replace the cation right to it. Pozolanic reactions between the lime and clay producing calcium silicate hydrates may also increase the soil strength. 1 - 2 % by weight is needed.

There is disadvantage that the stabilised soil becomes brittle and susceptible to breaking.

Ordinary table salt has also been used to stabilise dispersive soils. Chandra & Chen (Bergado.D.T (1987)) noted that 1 % sodium chloride gave maximum strength for the soil they investigated.

Chandra & Chen (Bergado.D.T (1987)) showed that the addition of 5 % flyash, a waste product from the burning of powerdered coal in thermal power plant, to dispersive soils yielded optimum strength. Pozzolanic activity increases the strength due to the calcium present.

9.0 FILTER DESIGN

Size of dispersive clays are very small (0.1 to 0.01 microns) as that they will not be caught in the voids of even a fine sand filter. However if the soil consists of clay composed of quantities of silt and dispersive fine with reasonable amount of coarse sand, the sand and silt could deposit on the filter and accumulate after the fine was washed away. As they are relatively impervious and also as dispersive clay has a tendency to swell, both these factors could seal off the leak.

If the impervious embankment comprised of sand and gravel and relatively small percentage of dispersive fines, this sealing action might not occur.

However these are tentative conclusions and researches are being done to develop filters on the basis of quantitative test results mentioned.

Recent laboratory research has shown that sand or gravelly sand filters with average D_{15} = 0.5mm or smaller will safely control and seal concentrated leaks through compacted specimens of the great majority of dispersive clays (with (D_{85} larger than about 0.03 mm). Sand filters with average D_{15} = 0.2 mm or smaller are conservative for the very finest dispersive clays.

10.0 CONCLUSION

High ESP values and piping potential are generally found to exist in soils in which the clay fraction is composed largely of montmorrilonite. Some illites (3 layer mineral) are highly dispersive. High values of ESP and high dispersibility are rare in clays largely composed of kaolin.

Crumb test gave preliminary indication of the dispersivity of the soil. However, pinhole and chemical tests are necessary to ascertain the dispersivity. From the 24 number of samples tested at the Open University, pinhole test results classify 03 samples as highly dispersive D_2 and percent dispersion for these samples were greater than 65. Crumb test results too classified those 03 samples as highly dispersive. However there were 02 samples with high percent dispersion which were classified as ND_1 by the pinhole test. A few samples were classified as slightly dispersive, ND_4 & ND_3. However these results should be confirmed by the SAR test as discussed earlier.

Atterberg limits test should be carried out to get some clue on the amount and type of clay present. It was noted that the 03 samples which were classified as D_2 were obtained from the dry zone of Sri Lanka which is in agreement with what was expected from Section 2.4 & Fig 3. Engineers should be warned about constructing earth dams in such areas and identification tests should be carried out. Special precautions should be taken to raise the reservoir level slowly after construction.

However research is needed on a large number of samples from dams of dry zone area which has substantial quantities of montmorillonite clay mineral and inspection of the sites is required.

The pinhole test is based on the subjective evaluation of the occurrence of dispersion. For soils with τ_c = 0, turbidity was greater than zero; the main reason being the qualitative nature of the test. However, this is a direct method.

Dispersion test gives a crude method of measuring dispersibility as no concern is given to actual existence of soil particles in the field. This does not tell either about the in-situ or intact structural status or does it take into consideration the interaction of pore and eroding fluid composition. However majority of samples shown to be dispersive by pinhole test are agreeing with this test. But vice versa is not correct.

ACKNOWLEDGEMENTS

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APPENDIX 1

PREPARATION OF SATURATION EXTRACT IN THE LABORATORY
(For the Chemical tests to find SAR & TDS)

About 250 gm soil passing 2 mm sieve is mixed with distilled water to consistency near the liquid limit and kept in an air tight container for about 24 hours. Pore water is extracted using buechner funnel, vacuum flask and vacuum pump as shown in the schematic diagram given in Fig.13.

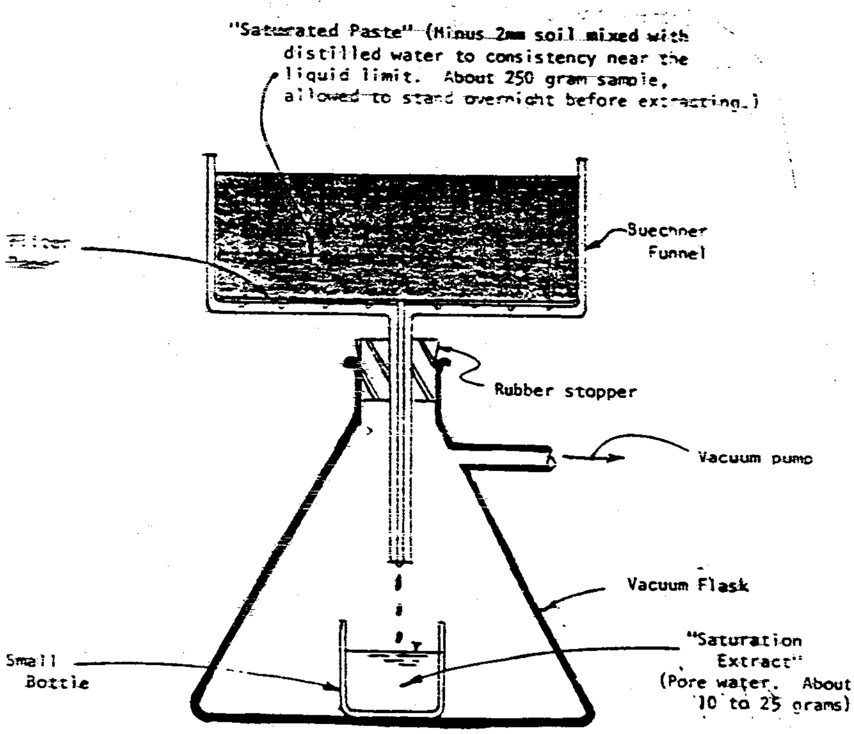

Fig. 13. Schematic Details of apparatus employed to obtain 'SATURATION EXTRACT" for chemical tests

A CASE STUDY OF SAMANALAWEWA DAM AND RESERVOIR LEAKAGE

BY VERNON F. PEREIRA, ENGINEERING GEOLOGIST,
CENTRAL ENGINEERING CONSULTANCY BUREAU, COLOMBO 07.

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A CASE STUDY OF SAMANALAWEWA DAM AND RESERVOIR LEAKAGE

BY VERNON F. PEREIRA, ENGINEERING GEOLOGIST,
CENTRAL ENGINEERING CONSULTANCY BUREAU, COLOMBO 07

SUMMARY

The Samanalawewa Dam & Reservoir Project encountered severe leakage problems even during the first filling. The reservoir was not therefore raised to Full Supply Level and the Project is due for major remedial works. Detailed studies were made during construction in order to ascertain particularly the watertightness of the immediate right abutment and saddles for cut-off works. These studies mainly involved 4800 m of drilling investigations to depths of the order of 200 m with piezometers installed. The piezometer monitoring within a few months revealed a flat groundwater table at river level responding to river fluctuations. Constructing a blanket on the invert and on the steep slopes of the reservoir in the suspected ingress area was considered at the initial stages for cut-off design. Subsequently, however a 105 m deep extensive grout curtain from an 1315 m long adit driven a little above river level was decided to be provided as a positive cut-off measure. Grouting from this gallery towards the top in a selective manner to elevations varying el. 420m to FSL at el. 460m was also done. The main curtain below the valley with 53,600 m drilling/13,450 tons of cement grouted with the maximum take of 15.5 tons/linear metre, proved ineffective and no hydraulic gradient could be created across it. The 1st trial impounding in June 1991 disclosed the pervious condition of the right bank with leaks occurring when the reservoir had barely risen to el. 398 m i.e. about 30 m above the river bed against the intended height of 90 meters to el. 460 m.

The second trial impounding commenced in March 1992. Potentiometric levels rose in the right bank ridge to the reservoir level within about one days time lag as earlier. Leaks slowly increased, progressively small earthslips occurred in the weathered mantle on downstream of dam along right bank. Water came into the dam cavity increasing the volume in the dam seepage measuring chamber. Also about 3 km far away in Killekandura ara on the left bank downstream of the dam a leakage of about 40 l/s occurred when the reservoir was around el. 424 m, noticed in June 1992 for the first time. The sudden burst 320 m downstream of the dam on the right bank around el. 400 m bringing out much more than 7 cumecs occurred in October 1992 when the reservoir was at el. 438.99 m; at the same time all the piezometers dropped drastically by almost about 25 m down to el. 415 approx. The discharge decreased to 2 m^3/s then the GWL rose again by 6-7 m presumably due to partial obstruction of the natural leakage privileged path around the hydraulic control section. It is observed that with the drainage which stays around 2 cumecs that the difference between reservoir water level and ground water level stays around 10 m for reservoir level variation from el. 426 m to el. 433 m so far. The point to be taken note of here is the fact that the long term average annual inflow at Samanalawewa dam is only 19 cumecs. And also note is to be taken of the fact that to what degree the natural unplanned but priviledged path developed during the burst can be relied upon as a dependable measure for leakage to keep the peizometric pressure lower than the water level in the reservoir.

Internationally reputed experts were consulted 1) to assess the stability of Samanalawewa dam, as built and under the present hydro-geological condition 2) to assess the stability of the right bank of Samanalawewa Reservoir under present and short term future conditions before remedial measures are complete and 3) to review the available proposals for conceptual design of remedial measures for the Samanalawewa dam and reservoir and to suggest the most appropriate proposal.

Concluding, the panel stated that the dam is safe for full reservoir and that work is required to reduce leakage for economic, not safety reasons. The dam and the right abutment rock just downstream of the dam, is safe against high abutment piezometric levels and drainage is not necessary. The unusual geological conditions make improvement of the curtain unpractical. The most appropriate measure with the probability of significant reduction in leakage is blanketing.

From October 1992 to date (Oct. 1993) the reservoir has been kept around el. 430 m (not letting it fill to el. 460 m, design TWL) in a controlled manner for partial generation of power. Geophysical investigation, seismic reflection survey on Samanalawewa reservoir to provide information on the ingress area/location of likely ingress areas in the reservoir have been conducted. Monitoring RWL, GWL and leakage goes on a regular basis. Ground water chemical analysis to identify leakage paths are also carried since July 1992 on a monthly basis.

This bulletin highlights the historical background of the project especially in connection with the right abutment/saddle investigations which has been addressed from 1958 onwards. It also presents the geological, hydro-geological features of the area and discusses about the proposed remedial measures.

Project Objectives

The main project objective was to meet the growing demand of electrical energy for Sri Lanka with an installed capacity of 120 MW. The Samanalawewa hydro-power project is designed to produce a firm 420 GWh of electricity annually and some secondary energy. A secondary objective would be the increase of the irrigation capacity of Udawalawe. Successful commencement of impounding of Samanalawewa was expected in January 1991.

1.0 Introduction

The project site is located about 160 km south east of Colombo about 10 km east of the town of Balangoda, in Sri Lanka. In the region of the project, the Walawe Ganga, its tributary Katupath Oya and the secondary tributary Diyawini Oya flow parallel in an easterly direction. At a point where the Walawe Ganga and the Katupath Oya come nearest to each other, a distance of a about 6 km, the natural water level in the Walawe Ganga exceeds that of Katupath Oya by about 300 m. The principal element of the project is to make use of the level drop for power generation at a station on the Katupath Oya (see Fig. 1).

The first stage development of the project consist of a zoned rockfill dam with a vertical centre clay core on the Walawe Ganga with a crest length of 530 m and a height of 100 m above river bed level, a gated spillway, a waterway system, 4.5 m diameter, 5.4 km long tunnel with an intake structure about 6 km upstream of the dam on the Walawe Ganga, a surface power station with an installed capacity of 120 MW (2*60MW), and a tailrace canal connected to the Katupath Oya. (see Fig. 1 and table 1).

It is envisaged that a second stage development of the project will comprise a dam across the Diyawini Oya, which is the river flowing between the Walawe Ganga and the Katupath Oya, an intake structure on the Diyawini reservoir connected to the surge chamber of the first stage development, a second penstock down the escarpment and an extension to the power station to accommodate two extra machines.

The project benefits from both the 'Yala' south west monsoon in the months from April to June and the 'Maha' north east monsoon in the months of October to December which is ideally suited for power generation the whole year through.

2.0 Hydrology

The Walawe Ganga is one of the major rivers in Sri Lanka, flowing southeasterly for a long distance at its early stages in the mountains, then northerly for a short stretch, and then back again in the southeasterly direction before it flows in the southerly direction to the sea. The catchment area of Samanalawewa dam site is 341.7 km². The discharge of the Walawe has been gauged and measured since 1958. The basin rain fall on the Samanalawewa catchment area has been available since 1921. The average annual rainfall in the basin has been calculated to be 2967 mm. The long term average annual inflow at Samanalawewa is 19 cumecs. (see table 1).

An estimated of 82575 m^3/year of sediment transport into the Samanalawewa reservoir has been expected. Studies of sediment levels over a 100 year nominal life have been reported to indicate a safe power intake tunnel at invert elevation of 417 m, 9 m above river bed level.

3.0 Previous Investigations and Reviews

The development of the Walawe Ganga basin has been studied at intervals since 1957 by the government of Sri Lanka. These studies have included detailed ground investigations with mapping, drilling, insitu permeability testing, exploratory adit excavations, geophysical, seismic and resistivity testing and material investigations (see table 2).

Ground investigations carried before construction are as follows :

(a) Two phases of investigations were carried out in the periods 1958-1960 and 1964-1966 under the direction of Engineering Consultants Incorporated (ECI), Denver, Colorado, USA.
(b) A limited amount of further investigations were carried out during the period 1972-1973 under the directions of the Snowy Mountains Engineering Corporation (SMEC), Australia.
(c) A further phase of ground investigations was carried out in 1975-1978 under the supervision of Technopromexport (TPE) Hydroproject, Institute, Moscow.
(d) Further additional geotechnical investigation for Samanalawewa dam was carried out by Nippon Koei Co., Ltd. (NK), Tokyo, in 1986-1987.

In addition to the reports, associated with the above investigations a number of review reports have been prepared since 1982, these did not involve any further field investigations. They are as follows :

(a) Reconnaissance report on Samanalawewa multi purpose dam project, prepared by Nippon Koei Co. Ltd.,Tokyo,1982.
(b) Samanalawewa Hydroelectric Project, Technical report, engineering review recommendations, 1984, prepared by Balfour Beatty Limited (BB), GEC Energy Systems Limited, Sir Alexander Gibb

and Partners (GIBB) and EPD Consultants Limited.

(c) Samanalawewa Hydro Electric Scheme, Review report 1984, prepared by Electrowatt Engineering Services Limited (EWI), Zurich, Switzerland.

(d) Samanalawewa Hydro Electric Project, Technical report, 1985 prepared by Central Engineering Consultancy Bureau (CECB), Colombo, Sri Lanka.

However it was recognised far before and at the time of award of LOT 11 construction contract at the end of 1987, that the following matters needed follow up in relation to the watertightness requirements for the reservoir.

(a) There remained uncertainties regarding hydrogeological conditions at depth in the right abutment. Additional drilling work was therefore programmed for an early stage in the construction contract with a view of establishing the limits for the grout curtain.

(b) Further investigation for the saddle areas, principally on the right bank was scheduled to be carried out by the Lot 11 contractor. (see Table 2 & 3).

4.0 Historical Background, Studies and Proposals in Connection to Watertightness

Identification and recommendations of the seven organizations in connection to the reservoir watertightness in the areas of the dam right abutment and saddles were recorded as follows. (See Fig.2 for location of saddles and table2).

ECI Study - May 1966

Geological investigations identified solution cavities in the right abutment of the dam associated with lenticular limestone bands. No special reference was made regarding reservoir watertightness. Drilling in the right bank saddles 1 and 2 looking for good rock for a natural spillway was not successful because of the poor quality rock encountered.

Forty five boreholes were done on the dam site adding to a total of 1710 m of drilling. The maximum depth drilled was 84 m. Four boreholes were done on saddle No. 1 (610 m from the dam) adding to 135 m of drilling. The maximum depth drilled was 39 m. Six boreholes were done on saddle No. 2 (1220 m from the dam) adding to a total of 369 m. The maximum depth drilled was 72 m. (see Table 2).

SMEC Study - August 1973

Three boreholes were done on the dam site adding to a total of 184 m. The maximum depth drilled was 77 m. One borehole was done on saddle No. 4 (2286 m from the dam) to a depth of 62 m.

It was stated that core drilling done by ECI has shown that the left bank area is satisfactory for the construction of an earth, rockfill or concrete dam, but the right bank has certain problems arising from the karstic nature of metamorphic limestone.

It was also stated that the saddles on the right bank are narrow and one of these has steep downstream slopes but the saddle on the left bank is wider at reservoir level. These saddles are generally at topographic elevations 15 m to 46 m above the reservoir.

Recommendations were made to obtain better core recovery using triple tube core drilling in the weathered rocks where the double tube core drilling has not given adequate core recovery. Continued studies of seasonal groundwater table fluctuations at the dam site and reservoir periphery saddles were also recommended.

TPE Study - 1978

Detailed geological and geophysical investigations were carried out on the dam site and some limited drilling investigations on saddle 2. The study identified that seepage losses from the Samanalawewa reservoir will take place only in the dam foundation, past its wings and in saddle no. 2 (1300m from the dam). Tentative calculations estimated that the seepage losses past the dam foundation and its wings without cut-off measures will be 150 l/s. In saddle 2 where the bed rocks are deeply weathered the calculations showed that at this reach 1.1 km long, the water losses can amount to 50 l/s without piping.

Fifty five bore holes were done on the dam site adding to a total of 2663 m. The maximum depth drilled was 200 m. The drillings included pumping in and some pumping out permeability test. Adit driving on the left and right banks and geophysical test, seismic and electric were also done on the dam site. In saddle no. 2, 4 boreholes were done adding to a total of 549 m. The maximum depth of any hole drilled was 150 m.

The TPE brief report recommended to consider at the next stage of design development the establishing of a network of piezometric holes to check seepage flow from the reservoir through the depression on the right shore initially to adjudge the response of the right bank area and ultimately to assess the likely volume of seepage when reservoir is created. It stated that the results of these observations will help to decide on the necessity of cut-off actions in this depression. (This valley is interpretted to be the right bank valley just downstream of the dam in saddle 1 area, See fig. 2).

BB/GIBB Study - April 1984

The engineering review and recommendations in this technical report were primarily a review of the earlier feasibility studies. BB indicated that their design given in the technical report was at a level between feasibility study and tender design.

Commenting on the ground water condition and rock mass permeability it stated that underdrainage of the hillside appears to be occurring. This is indicated by the loss of drilling water at depth in a number of boreholes, the falling water levels recorded as drilling progressed and enormously low piezometer readings. The underdrainage was thought to be primarily due to the presence of weathered limestone layers which provide a high permeability path to the river, both upstream and downstream of the dam site. Faulting may also play a part in under draining the right bank.

Commenting on the reservoir watertightness it identified that groundwater lower than reservoir top water level occurs at saddle 1 and saddle 2 on the right bank and at the saddle on the left bank 700 m from the dam, on the downstream side within 400-600 m of the reservoir margin. The geological structure is such that potential seepage paths exists along the strike of the foliation.

Saddle 2 was investigated during the period 1975-1978. Four boreholes indicated that zones of grade C material extended to depths of 120 m below proposed reservoir top water level. Losses of drilling water occurred overnight and there were zones where pressures during permeability testing could not be attained. Simplified calculations suggest that seepage losses of between 20-100 l/s could occur through this saddle under full reservoir conditions. It had Grade C material - Highly to completely weathered rock and residual soils.

No information regarding saddle 1 and the left bank saddle was available for this study, but in view of the conditions found at saddle 2 and at the main dam site it stated that karstic limestone bands or faults may provide seepage paths out of the reservoir area at these locations.

Further work at all three sites will be necessary to provide a sufficiently reliable estimate of seepage and to provide data for the design of any cut-off works.

EWI Study - November 1984

The Electrowatt Engineering Services Ltd., November 1984 report consisted of a detailed review and evaluation of the technical report on the Samanalawewa Scheme presented in April '84 by Balfour Beatty Ltd. in conjunction with Sir Alexander Gibb and Partners et. al.

With reference to the Samanalawewa dam it stated that a rockfill embankment with impermeable clay core is the best type of dam for the site. The overall dimensions and precise location indicated by BB for this structure can also be supported. Likewise it stated that they are in full agreement with the proposal of BB to construct twin diversion tunnels through the left abutment of the dam, rather than a single tunnel on the right bank, as selected by the Russian study.

It also said that BB rightly state that additional exploratory work is required to determine the precise excavation depth for the dam, unfortunately, the drilling technique applied previously does not permit a distinction to be made between soil, completely weathered, and moderately weathered rock. Most of these materials have been destroyed and noticeable core recovery (above 25%) usually starts only on slightly weathered to sound rock.

With regards to reservoir watertightness it stated that TPE presents a geological map of the reservoir which shows the general structure of the area concerned and the main tectonic discontinuities. Concerning watertightness of the reservoir and stability of its slopes no details are given.

It also stated that the general watertightness of the reservoir has never been questioned. The gradient towards other depressions are favourable, with the exception of three saddle zones two of them on the right bank and one on the left bank. Further investigations of these locations as proposed by BB will be essential to ascertain the stability and permeability of the potentially weak sections of the reservoir rim.

EWI Study Additional - August 1985

EWI commenting further on reservoir watertightness stated that four saddle ~~zones occur~~ and that they are located as follows :

Saddle 1	right bank 700 m south of the dam site
Saddle 2	right bank 1200 m south of the dam site
Saddle (4)	right bank 2100 m south of the dam site
Left Bank Saddle	1100 m north-east of the dam site

At these locations, ground level lower than the maximum water level is encountered on the opposite side of each saddle, within 400-700 m of the reservoir shore (See fig. 2). Saddles 1, 2 and (4) have similar geological characteristics and are situated along a steep ridge running north-south about 600 m east of the Walawe Ganga. The downstream slopes of these three saddles are steep and several small springs can be seen at about el. 440 m on both sides of the ridge. Opposite of saddle (4) on the left bank of the Walawe, a 20 m thick marble band outcrops, and shows karstic features. This cannot be traced through to the right bank, but the possibility remains that lenses of this probably boudinated layer could be encountered in the saddle zones.

No investigation data is available for saddles 1, 4 and that of the left bank but their geological conditions being similar to those of saddle 2 seepage loss must also be expected at these locations following impounding.

These four saddle zones are the only areas where significant reservoir seepage is likely to occur and further exploratory works are necessary, especially at saddles 1, 2 and (4) to provide reliable data for the design of any additional counter measures.

It further states that it is recommended that, in conjunction with final design, further exploratory core drilling be carried out at the dam site to investigate in particular along the right abutment, the nature and depth of loose overburden, highly weathered and disintegrated bed rock, and moderately to slightly weathered bed rock, as well as to outline the morphology of a rock face which is acceptable as foundation for the shoulders and core of the main dam. The drilling technique must be selected so as to permit the sampling of representative cores from the unconsolidated overburden and the assessment of the degree of weathering of the underlying rock.

Thirteen new holes with a total length of 1000 m in given locations were proposed to be done with water permeability test carried out in moderately weathered to fresh rock. The deepest drilled depth proposed was 120 m to be done on the right abutment from el. 500 m. In connection to the saddle areas it stated that they require further investigation with regard to the watertightness of the reservoir.

It was proposed that a drilling campaign, accompanied by water pressure tests be undertaken to assess the nature of the subsurface conditions and to obtain permeability data on the rock sequence concerned. Ten new holes of a total length of about 1150 m were proposed on all the four saddles. The proposed deepest depth to be drilled was 150 m in each saddle.

CECB Study - April 1985

The CECB study was on the basis of previous studies and reviews. With various reports and reviews in hand, CEB requested CECB to summarise, the various reports & reviews and to establish the project parameters and features based on the data available in these reports.

Commenting on the project it said that it was most suitable and economically beneficial. Commenting on reservoir leakage it stated that on the right bank it is more likely that some underdrainage takes place. It also further stated that seepage losses are likely to occur only in the dam foundation, two right bank saddles and the left bank saddle (located at 700 m from the dam).

The Project Off the Ground

The decision to start the project was taken in February 1986. (see Table 3). The design engineering contract with Sir Alexander Gibb and Partners (UK) came into effect in December 1986. The engineering supervision contract with Nippon Koei Co. Ltd. (Japan) and Electrowatt Engineering Services Ltd. (Switzerland) assisted by CECB (Sri Lanka) came with effect from January 1987. This group was called JVS which stands for Joint Venture Samanalawewa.

Lot I, diversion tunnels and relevant works contract came with effect from February 1987. The contract was completed in April 1988. The Contractor for Lot I was Hazama-Kumagai, Joint Venture, Japan.

Lot II civil works, dam and appurtenant structures which later included the right bank cut-off massive grout curtain after investigation during contract came into effect in October 1987. The Contractor for Lot 11 was Kumagai-Hazama-Kajima, Joint Venture, Japan.

Lot III civil works waterway and powerhouse came into effect in December 1986. The contractor for Lot III was Balfour Beatty Construction International Limited, UK.

Lots I and II was supervised by the JVS Japanese counterpart and Lot III was supervised by the JVS Swiss counterpart. The commencement of successful impounding of the Samanalawewa reservoir was proposed to be in January 1991.

NK Study - April 1986

The Nippon Koei Co. Ltd., Japan, study is reflected in their planning report of additional geotechnical investigations for design of Samanalawewa dam. Referring to the geological investigations the report stated, whereas the data of those previous investigations are useful for understanding of geological situations and problems of the site they are deemed yet insufficient for the detailed design purpose...... Water-table is irregular in shape at the right bank (abutment). Thus the limestone beds are presumed to cause leakage after impounding if no treatment is applied......... a deep weathered zone is found at the upper portions of the right bank slope, which might be caused by hydrothermal reaction.

A narrow ridge of the right bank continues southwards parallel to the Walawe river on which three saddle exist....... The limestone beds might if they occur, cause leakage through the ridge. Besides the said saddles may be formed by weathered, fractured and/or altered rock masses. The groundwater table in the ridge is not known at the moment. Thus leakage through the ridges especially at the saddle portion is to be carefully studied........ The firm foundation is to be detected by core drilling with water pressure tests. Rock material are to be recovered carefully by using elastic tube samples contained in a core barrel, especially for weathered and loose material portions, to know precisely the geological condition........ The drilling investigations.......... along the dam axis are urgently required for the tender design of the dam.

As for the limestone bearing bed on the right bank deep drillings with water pressure tests are to be performed to the level of el. 300 m in order to know the distribution of the limestone and caves therein. The piezometric head in the limestone, separate from the perched water is to be measured by using a partial water pressure measuring device, in several bore holes......... placed to obtain a hydraulic gradient at the right bank.

Four saddle portions, one at the left bank and three on the right bank ridge are to be inspected by drilling with water pressure test to a depth at which the ridge has a reasonable width of sound rock mass. The narrowest portion (saddle 2) is also to be inspected by an inclined borehole in combination with a vertical borehole, if a fractured zone occurs here. Those boreholes except the inclined one, are to be cased with a perforated PVC pipe in order to monitor watertable fluctuation before and after impounding....... The quarry site is to be studied by seismic exploration to know the thickness of sound rock without deep hydrothermal alterations.

Twenty six boreholes were proposed to be done on the dam site adding to a total of 1,630 m. The maximum depth drilled was to be 250 m. (J12, from el. 540 m to el. 290 m on the right abutment).

Four bore holes were proposed to be done in the saddles (saddles 1, 2, 4 and left bank saddle) adding to a total to 450 m. The maximum depth drilled was to be 150 m (in saddle 2).

Additional Geotechnical Investigation - July 1987

The July 1987, report on the additional geotechnical investigation for Samanalawewa dam is the connecting report of the Nippon Koei planning report.

The report stated that design of the project structures had also been made by the Consultant engineer Sir Alexander Gibb and Partners in 1985 with an implication that not a little more investigation would sill be necessary for further confirmation of the geotechnical condition. The additional geotechnical investigations planned by Nippon Koei Co., Ltd. was carried out in the period from June 1986 to June 1987. One of the major questions to be confirmed was conditions of weathering of the bed rock in the proposed dam site which would concern depth of foundation excavation for the dam and slope stability. Development and distribution of cavities in the limestone was another important subject. The work was performed by means of outcrop observations, core drilling seismic refraction prospecting and exploratory adit excavation.

Discussions were made with engineers of Sir Alexander Gibb and Partners for design in the course of the investigation for periodical evaluation of the investigation outcomes and review of the investigation program in the light of new findings. The investigation in this period is largely divided into two parts that is the investigations of dam foundation and the construction material investigation of embankment material and concrete aggregates.

Thirty nine boreholes were done during the 1986/1987 investigations. Those boreholes were limited to the dam site and quarry site. (See Table 4 for the proposed drilling investigations against the investigation done).

The report also stated that almost all geotechnical problems, left insufficiently answered after those massive investigations in the past were to be clarified or reconfirmed in these investigations.

This phase of investigations gave high core recovery in weathered rock. The bed rock on the right wing of the dam site proved to be intensively weathered to depths of more than 50 m or even 100 m, or to the level of elevation 420 m, some 40 m lower than the contemplated level of the dam crest. This deep intensive weathering seemingly due to hydrothermal alteration along faults is however located marginally off the foundation of the dam structure it stated. The same conditions for development of the intensive weathering seems to have worked also to accelerate the development of solution cavities in limestone....... The faults provide the routes for infiltration of weathering and hydrothermal alteration as well. Weathering of the bedrock in the dam site consists of ordinary weathering by air and water from the ground surface and a deterioration of rock through hydrothermal effect from underground....... There observed were a few cases of water losses, or rapid drawing-down of drilling water at some portions in boreholes where no openings larger than open cracks were found, in J12, at depth to 142.5 m at el. 399.62 m and in J38 between 152-154 m around ground el. 373.24 m.

The borehole J35 drilled at the location approximately 30 m downstream of the original dam axis on the right abutment proved that the depth of intensive weathering was not so deep....... the bottom of the weathering at elevation 445 m in J 35 as against elevation 420 m in J10. Considering the possibility that the foundation conditions may be better a little downstream of the original axis, an alternative dam axis was contemplated tilting downstream at an angle 20° from the original axis on the right bank. Three boreholes, J37, J38 and J39 were additionally drilled and seismic refraction was performed on the alternative dam axis, on which subsequently the dam was constructed. (see Fig. 5).

On the leakage through saddles it stated that in this additional investigation only a brief reconnaissance was made........ and a seismic refraction prospecting was conducted on a 440 m long traverse line at the saddle No. 2 location suspected for leakage from the reservoir.

Without the proposed drilling investigations having been done on the saddles, it stated that if there were any continuous cavities in limestone layers extending from the reservoir area to other basins, they would be the only sub-surface water passages to cause leakage from the reservoir. Considering that every limestone layer ever observed seem to change its thickness very often, it is unlikely for any solution cavity to continue for a long distance across the thick hills of watershed. However it is virtually impossible to prove, and the judgement is not more than a matter of probability. There are streams not lower than the reservoir level in the saddles No. 1 and No. 4 indicating that the groundwater is fairly high. The saddle No. 2 forms a narrow and straight dyke with two vallyes on both sides. According to the previous drillings, no limestone layer was recognized in this saddle. Accordingly it is deemed unlikely that these saddles might provide any substantial leakage paths, the report concluded.

The report also stated of one potential landsliding area located on the north-western slope of the right bank of the dam site at about 300 m upstream of the dam axis. (upstream of the confluence on the Walawe river).

Now stabilized on the steep slope the sliding mass seems rather thin. The present balance will very possibly be lost on impounding the reservoir to full capacity. Considering its location very close to the dam site, this sliding mass should be removed. 'Gibb' the design engineer studied this problem in detail and proposed to do nothing to the slope. The proposal was accepted by the Engineer.

5.0 Outcome of the Additional Geotechnical Investigations

As a result of the 1986/1987 NK investigation the original Russian dam axis was partly diverted curving it downstream on the right abutment because good rock was met at a higher elevation. The move indicated substantial savings in foundation excavation.

The unresolved uncertainties regarding watertightness of the right abutment were expected to be confronted by driving four grouting/ investigation adits at different elevations in to the right abutment along the dam axis. (see Fig.6).

The adits were placed between el. 463.40 and el. 384.63 invert levels spaced at an average of 26.25 m in the same plane. The excavated diameter was 4.10 m and the finished diameter was 3.50 m. Adit A with invert level 463.40 was designed to be driven 100 m and Adit D with invert level 384.63 m was designed to be driven, 309 m. A system of jet flush and grouting was introduced to be done between these adits to provide a positive cut-off.

Twenty metres downstream of the four grouting adits at around el. 411.63 m a single drainage adit of 3.5 m diameter running 182 m parallel to the grout curtain was designed at the on-set of the project and later cancelled. The drainage curtain designed incorporated 55 mm diameter perforated PVC pipes set at 5 m to 10 m intervals taken up to elevation 455 m and downwards to elevation 395 m. This drainage adit was not constructed.

6.0 Present General Layout

The Samanalawewa dam is situated on the Walawe immediately downstream of the confluence with the Belihul Oya. At the toe of the dam towards the left bank is the Mattihakka Ara. The valley widens from this point onwards. At the dam axis the valley is 60 m wide at the bottom and about 530 m at the dam crest elevation.

The Walawe valley within the area of the dam is a "V" shaped gorge with somewhat steep asymmetric slopes. Steepness of the right bank slope is 28° to 32° and that of the left is 23°-26°. The slope on the right bank rises to 530 m elevation and the slope on the left bank rises to 480 m elevation.

The Samanalawewa is a zoned rockfilled central vertical clay core dam. The deep weathering of the right bank was the main factor which influenced the type of the dam. The dam axis is curved, slightly towards the left bank and more towards the right bank which is convex towards the upstream. The dam has a maximum height of 100 m. The length of the dam crest is 530 m. The total fill volume is 4.48×10^6 m^3.

The dam's 1.1×10^6 m^3 central clay core is of residual origin. The material is classified as well graded clayey sand with a mean liquid limit of 43.2% and a plasticity index of 19.6%. It has favourable properties with regard to workability, compaction, settlement and imperviousness. Filler and transition zones from processed material were provided at the upstream and downstream faces of the core. Sound gneiss excavated from a special quarry was used for the rockfill shells. Rockfill material from the quarry which did not satisfy with regards to soundness and grain size distribution was built into the random rockfill zone. The quarry produced more such material than anticipated which led to a modified design to accommodate such. The lower permeability material required the introduction of a special drainage layer. For the core trench, excavation was done up to slightly weathered rock. For the shell area, excavation was done up to moderately weathered rock.

Comparison of Two Similar Types of Dams in Sri Lanka

	Samanalawewa	Randenigala
Crest Length (m)	530	485
Height (m)	100	102
Full Volume (Million m^3)	4.8	3.7
Upstream Slope	1:2.1	1:1.5
Downstream Slope	1:1.8	1:1.5

Dam rockfilled zoned with central vertical clay core

The river diversion during construction was designed for the 100 year flood corresponding to peak discharge of 2000 m^3/s. To reduce peak discharge considerably and economize the river diversion as high an upstream coffer dam of 35 m [403 m - 368 m] and 2 parallel diversion tunnels were constructed. The coffer dam was to be an integrated part of the main dam. The diversion tunnels were horse-shoe shaped with an inner finished circular diameter of 6.8 m. They were concrete lined along their entire length of 482 m and 502 m. With this design and taking account of the reservoir retention effect, the maximum discharge through both diversion tunnels could be reduced to 1450 m^3/s in case of the 100 year flood.

The bottom and irrigation outlet is designed for a maximum discharge of 80 m^3/s at retention level el. 460 m. It will serve two purposes first, in an emergency it will allow for a lowering of the Samanalawewa Reservoir to a minimum level of el. 372 m within a few weeks. Secondly it will be operated for a continuous release of irrigation water.

Seepage through the foundation beneath the river channel and the left abutment of the embankment is controlled by a conventional grout curtain taken into rock with low permeability. Immediately beneath the core of the dam, in the area of more permeable rock seepage is restricted by a concrete cutoff trench incorporating the grouting gallery which runs along the whole length of the embankment. In addition to being an important element of seepage cutoff works the gallery enabled the construction of the grout curtain to proceed independently of the construction of the embankment and provide access to the right abutment grouting adits. The concrete cutoff trench has a rectangular shape with a semi-circular roof. The width of the gallery is 2.5 m and the height 3.5 m. In the reach of the river bed consolidation/blanket grouting was performed on the bottom of the clay core foundation.

One of the most critical aspect of the design of the embankment was the limitation and control of seepage through the faulted, deeply weathered and karstified "limestone" foundation at the right abutment. A series of four grouting adits set below one another between el. 463.40 and el. 384.63 along the dam axis into the right abutment was designed to be driven at the very inception but partly abandoned after construction and the top most and bottom most adits diverted upstream towards the high groundwater table region to provide the necessary positive cutoff.

Seepage through the right bank saddles was designed to be controlled by the 105 m deep grout curtain drawn from the 1.3 km long, 4.1 m excavated diameter grouting adit driven approximately at el. 390 m.

The spillway is located on the left abutment of the dam, is of the gated overflow chute type discharging into the river, through a flip bucket. It is designed to pass the 1 in 10,000 year flood of 3600 m^3/s with gates fully open and the reservoir at top water level. The spillway discharges are controlled by three 11 m wide and 14.1 m high radial gates of the automatic type. The sill level is at el. 446.7 m.

The system of the transferring water from the reservoir to the power plant comprises:

* A power tunnel intake situated on the right bank of the Walawe about 5.5 km upstream of the dam site.
* A power tunnel 5.423 km long fully concrete lined with an internal diameter of 4.5 m.
* At its downstream end, the tunnel terminates at a portal valve house located on the face of the escarpment above the Katupath Oya. From this point a 840 m long steel penstock is installed down the escarpment to the power station. The diameter of the penstock vary from 3.85 m at the valve house to 2.85 m at the bifurcation at the power station.
* The surge chamber 18 m in diameter is located in rock at the top of the escarpment. It is ~~94 m in~~ height.
* The power station is located on the flat flood plain of the Katupath Oya, at the foot of the escarpment. The tailrace is connected to the Katupath Oya by a 530 m long canal. The layout of the power station is conventional for a surface power station. The power plant is equipped with two 60 MW units.

The civil works of the power tunnel and the power station done under the Lot III contract are funded by the UK, whilst the civil works of the diversion tunnels and relevant works done under Lot I contract and the civil works of the dam an appurtenant structures including the right bank cutoff works done under the Lot II contract are funded by Japan. (see Table 3).

7.0 Regional Geology

Geomorphology

The Samanalawewa project structures are located with in the so called middle peneplain which in the north is confined by the central highlands' main escarpment (the world's end escarpment) and in the south and southeast terminates in the Kaltota Hapugala escarpment and the vast flat land extending to the south coast of the island. The hilly surface of the middle peneplain, which rises above the coastal valley as a bench is about 300 m to 500 m high is dissected by river valleys and numerous ravines. Usually hills have steep slopes and form ridges extending along main tectonic structures. Hill tops are 150 m to 200 m above gorge bottoms.

The Walawe Ganga is the third largest river of Sri Lanka. It rises in the southern part of the central highlands. Along a considerable length from the very origin, the Walawe Ganga flows in a deep gorge to the east southeast. At Watawala the river turns at a right angles towards the north northeast and at the confluence with Belihul Oya it turns sharply again to the east southeast. In the valley the Walawe Ganga turns to the south southwest and does not change its direction right down to the ocean. The left tributary Belihul Oya and the right tributary Mulgama Oya, fall in to the Walawe Ganga within its middle stretch and in the valley the Weli Oya falls into it from the left and the Katupath Oya from the right. (see Fig. 1).

While the basin drops from the central highlands in a general southerly direction, the feeder streams of the main river have east southeasterly flow directions in the western parts and south southwesterly flow direction in the eastern parts. A prominent feature is that while the foot hills continue southwards at comparatively high elevations for kilometers on the western part of the Walawe basin, very flat plains having elevation of 120 m to 150 m are found in the eastern part of the basin. Two prominent directions are found in the Kaltota-Hapugala escarpment. These are the south southwest adjoining the Walawe and the east west along the Katupath Oya.

Climate

Geographically the Walawe Ganga basin is located in such a way that it is influenced by both the northeast and the southwest monsoons. Three zones can be distinguished in the Walawe Ganga basin in terms of rainfall. They are as follows:

* wet zone including central highlands
* dry zone, the low valley area
* intermediate zone which includes the area of the middle peneplain.

The bulk annual precipitation falls in periods between March to April and October to December.

Structure

The main structural element of the first order is the Balangoda syncline (see fig. 3), with respect to the project area. The syncline is characterized by the north west strike and the asymmetric inclination of the synclinal limbs. The dip of the south west limb is 35° to 40° and that of the north east limb is 25° to 30°. The fold bends gently dipping 15° to 20° to the north west direction. The synclinal limbs are complicated by low order folds and faults. Dislocations or a break in continuity are found in excavations, mappings, seismic surveys, airphotographs, satellite images and drill core chartings. The axis of the syncline plunges west northwest. It runs somewhat parallel to the upper reaches of the Walawe Ganga and passes through the north of Balangoda. The repetitive east southeast and north northeast directional trend of streams are due to the structural trend of the region.

Rock Types

The Samanalawewa project area lies wholly within Precambrian high grade metamorphic rocks. The rock types consist of the following :

* Charnockite, charnokitic gneiss, migmatized biotite charnockite
* Garnet biotite gneiss, granulite, khondalite, migmatized garnet biotite gneiss
* Quartzite, biotite gneiss, garnetiferous gneiss
* Calc garnulite impure crystalline limestone
* Pegmatites and quartz veins

which belong to the Kaltota formation. The formation is divided in to two groups, the upper and the lower. The dam and reservoir are located in the core of the synform or in the upper Kaltota group. The intake structure, the tunnel and the power station are located in the lower Kaltota formation.

8.0 The Geology of the Dam Foundation and the Right Abutment

The dam site is composed of Precambrian metamorphic rocks. The upper rocks of the synform are on the right bank and the lower rocks are on the left bank. The top of the right bank consist of interbedded, granulites, garnet biotite gneiss, garnetiferous gneiss with thin intercalations and bands of charnockites, biotite gneiss and impure crystalline limestone. The thickness of this formation is about 160 m to 170 m. This formation called the upper granulite bench by the Russians in 1978 was conveniently divided to three other groups on the basis ofcharnockite and "limestone" occurrence by the engineer and designer. The groups were named as GRA 2, GRA 3 and GRA 4. The formation underlying the above said, which is mostly seen on the right bank and on the left bank on the diversion tunnels intake portal consists of interbedded and intercalated garnet biotite gneiss, garnetiferous gneiss, charnockite and impure crystalline limestone. The predominant rocks are charnockite. 'Limestone' is about 30% in this formation. This formation in the present context is named as CAL which was then called the upper bench containing carbonates. Lying below this formation is a marker which is a typical rock with very coarse grain garnets. It is a granulite which is 20 m to 25 m thick. The formation is seen in the lower part of the right bank and on the upper parts of the left bank towards the downstream. This marker band is called GRA 1 in the present context which was earlier called the lower granulite bench. The formation underlying the marker which was earlier called the lower bench containing carbonate is now named CHA 1, consist of alternating charnockite and garnet biotite gneiss with subordinate layers of impure crystalline limestone bands. The thickness of this formation exceeds 150 m. This formation is seen on the lowest part of the right bank, on most of the downstream part of the dam excavation at river level, on most part of the spillway excavation and on the upper parts of the left bank towards the downstream in the quarry area. Just underlying GRA 1 is a 200 mm to 500 mm thick band of "limestone" which is more or less highly to moderately weathered and even cavitated at places. The nature and association of this band with GRA 1 was found all over even in the investigation boreholes done on the right bank saddle 1. (see Figs. 4, 5, 6 & 7).

The dam site is located on the north eastern slope of the Balangoda syncline. On the left bank slope the rock is characterized by southwestern dips at an angle of 10° to 30°, on the right bank with southwestern dips at angles 25° to 40°. In the river the dip angles are around 5° to 10°. At the foot of the right bank towards the hill two small folds with axes northwest - southeast running sub-parallel to the river were encountered. (see Fig. 6). Seven major sub vertically dipping brittle-ductile tectonic dislocations or faults, slickensided, striated running in the north east - south west direction were encountered in the dam site excavation. The width of these strike - slip faults varies from 1 m to 5 m as encountered. All major faults were associated with slickensided closely to moderately spaced joints. Faults were observed to grow thick and thin in fracture intensity, to branch out like veins of a leaf running sub parallel to the main fault direction.

The intensity of fracture of the same fault varied very significantly when encountered running through different rock types. The fracture intensity was highest in charnockites and lowest in impure crystalline limestone. The largest measurable fault observed in the dam excavation was traced on the left bank cutting across the dam axis. The key marker granulite indicated a 10 m apparent vertical displacement at chainage 170 m on the dam axis. In the exposed excavation the most prominent and numerously found tectonic joint set was sub-vertical, striking north east. The other set which occurred less frequently was dipping vertical and striking north west. The Walawe Ganga valley within the area of the dam is almost a strike valley running sub parallel to the regional rocks. The shallow ravines on the slopes of both banks more or less are controlled by the north eastern faults as it is revealed in the excavation.

On the right bank within the CAL formation five considerably large intricate solution cavities, cavity No. 2 to No. 6 containing soft material were encountered. These solution cavities were cleaned of their soft material as far as possible digging in and were plugged with concrete and grouted. A naturally exposed solution cavity within the same CAL formation located 200 m from the confluence up the Walawe was excavated during TPE, 1978 studies. The cavity (No. 1) was plugged with concrete and grouted at the very end and the rest was filled with rockfill.

9.0 <u>Investigations during Construction</u>

In the middle of 1988 excavation of the grouting adits C and D in the right abutment encountered a substantial area of fractured rock that was apparently extending significantly beyond the limits. Further investigations therefore were necessary to establish the requirements for ensuring satisfactory arrangements for reservoir watertightness. Four exploratory holes D1 to D4 were drilled from the end of Adit D (el. 389 m) the lower most, before access was lost to the adit, These holes indicated fractured rock. Piezometers installed in three of these boreholes showed a groundwater level at el. 379 m. Permeabilities as measured gave Lugeons values of 5 to 8D. The design criteria used for the dam did not permit the grout curtain to terminate in the ground investigated by the D series holes. (see Fig. 6).

In addition excavation of the dam foundation revealed caves at upstream of the core trench at el. 420 m and at downstream of the core trench at el. 438. A cave was also revealed in the grouting adit C at el. 410 m. All these caves occur (and later 3 others were encountered in the dam foundation excavation close to the clay core trench) in the same geological unit (CAL) as the cave on the Walawe Ganga upstream of the dam site which had been located during the Russian investigations.

With the objective of assessing the requirements for extension of the right abutment grouting adits three deep boreholes, GW1, GW2 and GW3 from the surface beyond the limits for the grout curtain were then drilled. (see Fig. 5). These 200 m and more deep boreholes (done in Saddle 1 area) revealed continuing fractured rock and high permeability. In addition the holes showed that there was a perched water table relatively near the surface and that the true groundwater table was at around river level at el. 380 m as revealed by the installation of piezometers. The features identified were indicative of potential seepage paths at depths within the right bank as suspected before tender. It was therefore clear as earlier proposed that further study and investigation works were required for the development of the requirements for the dam cut-off works and drainage facilities for the right abutment and for investigations of the conditions at depth within the saddle areas in the vicinity of the dam to establish whether and what cut-off measures were required in those areas.

As extensive investigation works were carried out prior to tender for the embankment dam site as recommended, conditions as revealed by the excavation for the dam foundation were generally in line with expectations.

On the other hand, considerable difficulties had been encountered with the drilling of deep boreholes in the right abutment during the 1986 - 1987 pre-tender investigation programme. It was therefore decided to carry out some of the additional drilling work from the end of the lowest grouting adit (Adit D) which had been scheduled for excavation at an early stage in the construction programme. Limited additional investigation works in the saddle areas have been foreseen for execution by the Lot-II Contractor. This work amounted to 1,200 m of investigation drilling. 9,900 m of drilling for grouting, 500 tons of cement for grouting.

From July 1988 to June 1990 during construction in the dam right abutment and in all the saddle areas extensive rotary core drilling investigation with permeability, water pressure test, and piezometer installation was done. In all, 30 cored holes were done (five D series four MS series and 21 GW series which included 14A & 15A). The maximum depth drilled was 230 m. The total cored drilling was 4806 m. The deepest elevation reached was to el. 208 m by D5 cored from the grouting adit D at around el. 388 m. The hydrogeological study with the aid of piezometers proved very effective revealing two distinct ground water tables. (see Fig. 8). One was observed to be a relatively stable groundwater stable at around elevations 400 m - 410 m as depicted from the piezometers installed in GW9 and GW10 situated on the right abutment hill. The other was observed to be a flat groundwater table at around elevation 379 m as depicted from the piezometers installed

* in D1, D2, D4 and D5 cored from adit D
* in MS1, 3 and 4 done in the geomorphological depression of the right bank downstream of the dam
* in GW1, 2, 3, 4, 5, 6, 8 and 17 done in the saddle 1 area
* in GW 7, 13, 14 and 15 done in saddle 2 area
* in GW16 done in saddle 3 area and
* in GW18 done in saddle 4 area.

The RS series boreholes done during the Russian investigations in 1978 and J38 & J12 boreholes done during NK investigations in 86/87 indicated low groundwater at el. 375-380 m. No piezometers were installed in these boreholes.

Piezometers for the first time were installed in late 1988 early 1989. (apart from the limited number of piezometers installed by the Russians in 1978 in the dam area).

It was only in May/June 1989 that it was first identified by the Designer that the piezometers with the flat groundwater levels (around 379 m) reacted to the changes in river level with a delay time of around 24 hours. The piezometers in area B all behave similarly regardless of the elevation of the sealed response zone (between el. 398 to el. 208) and regardless of the location of the response zone be it above or below whatever the rock unit eg. GRA3. No apparent time lag between piezometers and no apparent hydraulic gradient through the right bank has been identified. From this information cut-off work on the dam right abutment and between the dam site and saddle 2 to control significant leakage was decided.

In an attempt to identify and locate cavities at depth during construction a trial seismic reflection survey over a 1640 m of traverse was carried out in the saddle areas. The low groundwater table and large thickness of weathered ground proved the technique to be unsuccessful for this purpose. Seismic refraction surveys between boreholes to identify potential targets for further investigations were then performed. Borehole GW 17 was done as a result to identify fractured rock in the saddle 1 area. A total of 3,700 m of seismic refraction survey was done.

From the investigation consisting of drilling and geological mapping carried out at the dam right abutment to the right bank saddles, the broad metamorphic stratigraphy recognised before construction continued to be sub-divided and added up further. The upper granulitic bench was further sub-divided in to GRA2, GRA3 and GRA4. (See table 5). GRA2 and GRA4 is more or less similar with predominently garnet biotite gneiss with some, charnockitic gneiss and thin limestone bands. Amphibiotite was sometimes encountered in GRA2 and GRA4. GRA3 similar to GRA1 is predominantly garnetiferous granulite, with no 'limestone'. GRA3 was designated as the impervious membrane which was supposed to be the least pervious of all units. CHA2 the unit containing predominantly charnockitic gneiss and some contorted limestone bands and GRA5 the predominantly garnetiferous granulitic gneiss were the other units encountered (see Fig. 5).

Based on the geological, hydro-geological, weathering and alteration observations made during drilling investigations during construction a sub-division of the ground beyond the right abutment to saddle 4 was identified. This area was divided in to two areas designated as area A and area B. Principally based on the presence and absence of calcite in impure crystalline limestone bands.

In area 'A' the mineral calcite in Impure crystalline limestone is present. Caverns and cavities in the CAL unit are broadly attributed to the work of surface water and air in circulation. Cavities at depth in rock units are also attributed to the above mechanism associated in the neighbourhood of faults and joints.

In area B the calcite of the impure crystalline limestone has been removed by solution in addition to the general mechanism possibly by hydrothermal action. Corrosive features and secondary deposits are prevalent in this area.

Other associated features encountered in area A & area B are presented in table 6.

Hydrothermal action in the right abutment and saddle areas is suspected to have occurred, ever since NK in April 1986 reported it's doubts about the phenomenon. X-ray analyses done on the minerals of samples obtained from the dam and saddle areas in July 1989 have identified certain minerals (anhydride, smectite and sericite) which could be the product of hydrothermal alteration considering the other related features like alterations and corrosion observed in the drill cores.

Major faults zones encountered in the dam site excavations were the NE-SW strike slip, 1 m to 5 m wide, tectonic dislocations. The prominent joint set was sub-vertical striking NE-SW. The other joint set encountered was in the direction NW-SE. The prominent geomorphological feature traced downstream of the dam in deep ravines on both gorge slopes also runs in the NE-SW direction. The clear lineation identified in the aerial photographs what is called the Matihakka fault. Investigations during construction supported mainly by hydrogeological data with groundwater contours identified a NNE-SSW Mattihakka type fault which demarcates areas A and B. The NNE-SSW, Mattihakka type fault is designated F1 type in general. (see Fig. 5 and 8).

The other type of fault identified are the EW sub-vertical faults which run sub-parallel to the Balangoda synform axis running across the neighbourhood of saddle 2. (F2 and F3 faults identified on site are EW faults). (see Figs. 3, 4 and 5).

Low angle thrust faults with breciated rock fragments of a 'cold brittle ductile nature' striking NW-SE thrusting NE wards are also encountered in the area. These faults seem to be associated with impure crystalline limestone horizons cross cutting foliation. This type of fault is not well understood. These are encountered in the road cuttings downstream of the dam and in the right bank cutoff grouting adits and in the right abutment cutoff adits.

Based on the geology geomorphology and hydrogeology of the dam right abutment and right bank saddle areas three main zones of potentially high leakage or privilege paths were identified.

(1) Through the Area A which is strike and fault controlled (the inclined thrust fault subparallel to the strike of the rock). The pervious zone along strike in the area lying between the upper unit CAL and the lower unit GRA2 was the vulnerable zone.
(2) Through saddle 1 along the NNE-SSW strike slip fault 1 cutting across the strike of the rocks which marks more or less the boundary of areas A and B and
(3) Through saddles 1 and 2 along the strike of CHA2 and GRA3/GRA4 boundary in association with the EW strike slip faults F2 and F3 which runs sub-parallel to the strike of the rock.

The Engineer considered that the potentital leakage path is along strike principally through karstic rock units. The Designer considered that potential leakage was both fault controlled and stratigraphically controlled.

Without any cutoff measures the Design Engineer's estimate was that the leakage would be in the range of 1 to 6 cumecs. This level of leakage would be unacceptable for the project. The Engineer estimated the leakage would be in the range of 1 cumec, considering the average permeability to be in the order of 5×10^{-3} cm/sec. The Design Engineer considered 5×10^{-3} cm/sec to be a 'lower bound' permeability and considered the ground very permeable similar in nature to fine gravel/coarse sand. The bulk permeability of the previlege path was estimated to be $1x10^{-3}$ cm/sec.

10. The Dam Right Abutment and the Right Bank Cutoff Grout Curtains

Further investigations upstream on the immediate right abutment of the dam by boreholes GW9, GW10 and GW11 established the higher ground water table (el. 400 m - 410 m) encountered by the Russian investigations. The weathered cavitated layer between CAL and GRA2 proved to be thining out towards the upstream. The right abutment cut off works designed to be done from the four adits A, B, C and D were abandoned beyond chainage 601 m, the section which ran into pervious ground up to chainage 727 m. The top most adit A and the bottom most adit D was diverted almost through 90° upstream to tie up the dam grout curtain in area A where there is a high groundwater table and good rock at depth. Adit Ab was driven 100 m and Adit Db was driven 300 m. (see figs. 6 and 7).

Leakage route 1, the leakage path in area A was intersected by adits A(b), D(b). Adit A(b) encountered highly weathered to completely weathered rock. Adit Db encountered 1 to 2 m wide subvert faults (NNE-SSW) around chainages 90 m, 110 m and 223 m. Completely weathered (sheared) zone was encountered between chainage 150 m and 163m, moderately to slightly weathered sheared along the foliation resembling a biotite schist was encountered between chainages 163 m and 236 m. The total drilling for grouting from adits A(b) and D(b) was 15,649 m. Altogether 1254 tons of dry cement was pumped into this area. The average grout take was 80 kg/m.

The total drilling for grouting along the dam axis between chainage 0 to 612m, was 15,540. The total consumption of dry cement was 71 tons. The average grout take was 4.6 kg/m. The primary holes were spaced at 4 m. The cutoff was set at 3 lugeons. (see table 9).

During the design stages of the right bank cutoff works the perviousness of the unit GRA3, 'the impervious membrane' which consist of garnetiferous granulitic gneiss devoid of impure crystalline limestone layers was much debated. in the Designer's view GRA3 offered the best potential barrier to seepage but said that it is most probably breached across strike at one or more zones by tectonic features permitting water to pass through it (leakage route 11). In the Engineers view the only conceivable breakage of Granulite 3 disconnecting it completely was across Fault 2 near saddle 2 (leakage route III). (see fig. 4).

Upstream of GRA 3, that further leakage paths exist across saddle 2 (leakage route 111) through the units known as CHA 2, GRA 4 and the faults F2 & F3 was the view of the Engineer as well as of the Designer.

It was also the view of the Engineer and the Designer. that highest permeabilities existed around levels el. 380 m and el. 300 m, the evidence extracted from the permeability, water pressure tests done in the drill holes during investigations.

As the Engineer's opinion was that GRA3 was breached only across Fault 2 and that the very pervious zone was horizontally around el. 380 m, grouting across fault 2 and CHA 2 from an adit at el. 390 m to el. 290 m was thought to prevent potential leakage to an acceptable extent. (see Fig. 5).

The designer supported by the groundwater contours (see Fig. 8) regarded the principal leakage to arise from NNE-SSW vertical faults (F1 type) breaching GRA3. As such proposed the cutoff across fault F1 as well.

Finally the right bank cutoff works was constructed from a continuous adit system with invert level close to 390 m, starting from saddle 2 aligned across leakage route III, across fault F1 area, leakage route II, and linking to the dam right abutment adit Db, leakage route I area cutoff. (see Fig. 5). The adit across fault F1 (called the H adit) was located with the intention of meeting the central part of granulite 3 at the level around el. 300 m. Effective grouting from el. 390 m through the el. 380 m in GRA 4 to the el. 300 m in GRA 3 wherever breached was thereby anticipated.

The right bank grouting adit 3.5 m in diameter concrete lined was 1315 m in length up to the adit Db dam right abutment cutoff adit. The continuous adit system is self draining through the dam adits Db and Da is driven above the existing groundwater table.

Fault zones were encountered between CH700 and CH820 for fault F1, between CH330 and CH400 for fault F2 and between CH120 and CH145 for fault F3. Fault F1 appears to have a considerable apparent vertical displacement disconnecting the rock units completely. Rock encountered from CH0 to CH60 was kaolinized garnetiferous garnulitic gneiss, from CH60 to CH420 was moderately to highly weathered predominently charnockitic gneiss with bands and zones of highly to completely weathered contorted/brecciated rock, cutting across the foliation, from CH420 to CH900 was predominently garnet biotite gneiss with charnockitic gneiss bands slightly to moderately to highly weathered and with completely weathered zones and bands of contorted/brecciated rock. From CH0 to CH910 practically all discontinuities were open. From CH910 to CH1315 the rock encounter was fresh and massive consisting of (migmatized) garnet biotite gneiss or garnetiferous granulitic gneiss, with the exception of greenish grey cavernous impure crystalline limestone encountered in association with a NE-SW slickensided faulted zone between CH1110 and CH1125. The width of the solution cavitated 'limestone' was about 5 m. Two other minor bands of 'limestone' were also met within this area.

The grout curtain which is not acting as a positive cutoff was designed to cross the privileged irregularly developed solution features connected with open discontinuities, was limited to a depth of 105 m below the grouting adit. The grout curtain was also anticipated to be hanging. The grout curtain was terminated at the southern extremity in GRA5 although beyond this point (GW16 and GW18) was responding to river water levels.

The conventional split spacing method was used with the primaries spaced at 16 m centres. In general cement bentonite grout with the following component : (0.7 W/1.0 C/0.02 B) was pumped in. The grouting pressure was 2 MPa for all 10 m stages except for the first 5 m stage behind the adit lining where it was limited to 1 MPa. A total of 53,200 m has been grouted with 13640 tons of dry cement i.e. an average of 256 kg/m (600 litres/metre).

Only very few spots were grouted above the adit up to el. 424 m and rarely to el. 460 m (see Fig. 9). Certain sections between CH 1200 and CH 1315 have not been grouted being considered impervious. The zones of high grout take were checked with cored holes and water pressure tested. In the curtain length from CH0 to CH1188 which has been treated

Sextary holes (0.50 m spacing) represent 2%
quinary holes (1.00 m spacing) represent 17%
quarternary holes (2.00 m spacing) represent 34%
tertiary holes (4.00 m spacing) represent 27%
secondary holes (8.00 m spacing) represent 25%

Seven cored holes have been done to a depth of 180 m below the adit (to el. 210 m approximately) shows that fractured rock, solution paths or cavities, corroded and altered rock still exist at these depths. From the primary hole grout take 500 kg/m configuration it seems that the curtain is hanging "in at least sections CH 0 - 40, CH 110 - 180, CH 430-500, CH 620-870 which is more than anticipated from the investigations (total "hanging" length of 430 m compared to less than 200 m estimated before grouting). The depth of grouting was decided to be a fix 105 m irrespective of the final stages grout-take. The curtain even with GRA3 encountered around el. 300 in the final stage seems to be hanging. It also seems that GRA3 as expected to be encountered around el. 300 m was not encountered in the last 150 m (between CH430 and CH280 in the H adit section of the cutoff curtain).

It was noted that right throughout before and after the right bank cutoff's massive grouting operation no piezometer response across the grout curtain was observed according to expectations. The only conclusion was that the grouting operation plugged only major holes to reduce leakage which was still at un-acceptable level.

11.0 First Trial Impounding and Observations

The first 'Trial Impounding' was on the 2nd June 1991. The impounding was on a condition to keep the reservoir level at el. 395 m using the low level outlet as flood control. The level el. 395 m was established considering, the incomplete grout-curtain in the immediate neighbourhood of the dam on the right abutment and in the cutoff works for the right bank and the incomplete concrete lining of the 290 m long access adit E driven in completely to highly weathered rock.

The 'Trial Impounding' was the 'acid test' of the engineering implications of the boreholes piezometers potentiometric level fluctuations with the river, the extremely high grout takes running into a maximum of 15 tons per linear metre and the ubiquitous drilling water loss in the investigation boreholes and in the grout holes of the cutoff curtain. Monitoring of the reservoir leakage was programmed before the trial impounding. The programme comprised of -

i) daily potentiometric level measurements of the piezometers installed in the 23 surface boreholes on the right abutment and on the four saddles on the right bank.
ii) water flow, weir measurements on the Kalunaide ara running downstream of the reservoir to which the saddles are exposed and on the right bank depression just downstream of the dam.
iii) observation points such as springs on the right bank of the Walawe downstream the dam up to Kalunaide ara, and on the left bank upstream of the Kalunaide ara.
iv) observation of the grout holes and investigation holes of the right bank cutoff adits and
(v) reservoir level measurement.

The reservoir level recorded the maximum level of 402.40 m on the 14th June within 12 days of impounding rising about 32 m above river bed level. Due to the unexpected adverse weather conditions, maintaining the reservoir level at el. 395 m was impossible even with the low level outlet fully open.

Immediately after the impounding what was most alarming was the quick harmonious response of the potentiometric levels of 16 piezometers in the boreholes scattered on the right bank from the edge of the right abutment just downstream of the dam axis to as far as saddle 4 which is about 2.4 km from the right abutment in area B. With respect to the right bank cutoff grout curtain 3 of these piezometers are located upstream and 12 of them downstream. One of them is located in saddle 4 about 1.2 km far from the end face of the cutoff adit. The deepest response zone of these piezometers as in borehole D5 drilled from the lowest grouting adit is between elevation 255 m and 208 m.

The other striking observation with the rising of the reservoir level was the development of the new spring on the right bank of the Walawe 320 m downstream of the toe of the dam. The spring was noticed for the first time on the 11th June just 9 days after impounding. The spring flowed from underneath an ancient landslide scar. The leakage measured at the beginning was 5 litres per second when the reservoir recorded 398 m and the average potentiometric level in the boreholes at el. 393 m.

Other major leaks observed were, from all the grout holes of the cutoff adit which is driven from around elevation 390 m, from the investigation boreholes done in adit D and from a construction joint in Adit D's concrete lining. The leakage water observed in the cutoff adits gradually increased as the reservoir level kept on rising. The maximum water leaked from the ancient slip area was estimated to be around 20 liters per second. And the maximum water leakaged from the D3 horizontal investigation boreholes in Adit D was estimated to be about 15 litres per second. The permeability estimated from the data obtained from this bore hole (D3) was 3.3×10^{-3} cm/s.

Other minor leaks in the stream near the toe of the dam were also noted.

A hissing noise with air emanating had also been recorded in the main power tunnel intake area after impounding. A similar phenomenon was also observed in some of the grout holes in the cutoff adit. The groundwater table in RT-1A a borehole 140 m from the intake gate shaft downstream along the tunnel trace read el. 380 m before construction. It was therefore suspected whether the flat groundwater table on the right bank was as far as the intake.

Drilling and grouting from late 1990 to 15th July 1991 for the dam right abutment cutoff from adits Ab and Db and for the right bank cutoff from the 1315 m long adit with invert level at 390 m compared to the final March 1992 quantities are on follows. (See table 7).

The quantity of drilling and grouting by the first trial impounding from adits Ab and Db was about 78% of the revised target and from that of the right bank cutoff adit was about 22% of estimated quantity. The grout take in the right bank cutoff was 467 kg/m while the initial estimated quantity was 100 kg/m.

Drilling and grouting of the right bank cutoff by the first impounding had accomplished less than half the work originally foreseen as necessary to achieve a satisfactory cutoff. When completely done to design it was anticipated to reduce leakage through the right bank to an acceptable level. However it was the opinion of the Designer that it was possible that even after completing the grouting, leakage may still be unacceptably high.

12. Right Bank Cutoff the Other Approach

Another approache such as lining part of the reservoir had been studied earlier but considered to be impractical. It was not possible to specifically target the areas for treatment by blanketing because of the lack of a well-defined hydraulic gradient in the right bank which made it impossible to locate the potential reservoir water ingress areas. There was also no evidence that leakage would be confined to the river. Consequently the area requiring treatment included the steep reservoir slopes where construction would be very difficult. In addition, to enable the construction of the blanket to proceed in dry conditions, extensive river diversions would have been necessary. Initial estimates indicated that to cover the entire potential area of reservoir water ingress, with an impermeable blanket some five million cubic metres of material might have been required. The proof that the treatment was successful would only have been obtained after the first impounding of the reservoir.

13.0 The Second Trial Impounding

The second trial impounding commenced on the 5th March 1992. The total quantity of drilling was 53,293 m and the cement consumption was 13,641 tons in right bank cutoff. The curtain kept to a, maximum depth of 105 m (to el. 285 m) from the grouting adit at el. 390 m irrespective of what ever was the quantity of grout take at the last stage. The piezometers located both downstream and upstream of the curtain and below the curtain showed the same flat groundwater table responding fast to the reservoir level changes.

Although an updated quantity of drilling and grouting was provisionally agreed in August 1991 there was a wide variation in estimates of the required quantity for grouting between the Designer and the Engineer for the right bank cutoff. The Designer's estimated quantity of drilling for grouting was 68,000 m against the Engineer's estimate of 45,000 m which was finally agreed to with the provision that reservoir impounding will not be done until the tests indicate an acceptable efficacy of the cut-off works.

The Designer's estimate of 68,000 m included 4,800 m of fan grouting at the southern end of Adit F, 4000 m of 150 m deep holes down to el. 250 m the fault F1 area and 14,200 m of 110 m deep holes for CH 260 - CH 620 in addition to the general upgraded quantity of

45,000 m in all. Time constraints, technical consideration and budgetary constraints were fully taken into account on the ultimate estimate of drilling for grouting which was, as explained above, essentially provisional in nature for planning purposes.

Follow up provisions, were allocated preparing for the event of seepage losses being found to be unacceptably high. Tentatively 5,000 m of drilling with 2,000 tons cement take was roughly set aside for this.

Broad guide lines of leakage monitoring was set by the Engineer accounting for the downstream requirement. (See fig. 10). Upto 500 l/s of leakage at el. 460 m reservoir top water level was to be acceptable leakage. Between 500 l/s to 2000 l/s of leakage at el. 460 m was to be the range of marginally acceptable leakage. Beyond 2000 l/s at el. 460 m was unacceptable leakage.

Events after the second trial impounding were as dramatic as for the first. All surface piezometers in area B of the right bank, practically all piezometers downstream of the grout curtain, practically all deep piezometers under the grout curtain and all piezometers in the abandoned adit responded to the reservoir level within about a day's time lag. Some piezometers in the dam right abutment cutoff were also noted to be responding to the reservoir water level. Weir measurements across streams and springs were also noticed to be increasing with the reservoir level.

Wet patches to leaky patches along weathered limestone bands of the downstream side continued to rise in elevation level (such as in the limestone band under GRA1.) with the reservoir.

As the reservoir was being impounded stabilization measures were undertaken in the slip area downstream of the dam where the first leak appeared during the first trial impounding. Perforated 80 mm diameter PVC pipes wrapped with mesh were installed into 110 mm diameter holes driven into rock. The horizontal distances driven varied from 11 m to 53 m. These drainage measures were take to stabilize the slope against the rising pore pressures which were visually manifestating. One of these horizontal drainage holes B5 driven around el. 381 m, met water gushing out at 11 l/s at distance of 43m (~ on 14/06/92). This hole was driven to 53 m and grouted. All the 53 m was in soft ground. This hole was close to the location where the burst occurred later in October 1992.

In the ancient slip area, a superficial slip occurred between elevations 402 m and 416 m when the reservoir was at el. 422.62 m. The area affected was 30 m x 60 m x 1.5 m. Water emerged at 6 l/s from this area. The first trial leak area was exposed excavating further into the hill, traced the leak to be emerging from cavernous 'limestone'. When the superficial slip occurred, the leak from the open solution cavity was around 65 l/s which was then the only major source of water loss. Nine RI series cored bore holes were done in this area. The total depth cored was 430 m, the maximum depth drilled was 70 m. Fresh 'limestone' was found in some of the boreholes of this area which indicated that this could be classified as area A. The collapsed material in this arm chair like depression was barley seen. The geomorphological depressional feature is more probably due to the differential weathering of limestone in this area.

Two hundred metre downstream of the toe of the dam on the right bank and 110 m downstream of the access adit Da is located MS 1 with collar at el. 415 m. In May 1991 as reservoir level moved upto el. 415 m and above, water from MS 1 flowed out like from that of a fountain adding to the knowledge of the perviousness of the ground just downstream of the dam, in the valley opposite the Mattihakka Ara valley.

The piezometer located in cavity no. 4 in the dam cavity at el. 401.8 m, 23.31 m downstream of the dam axis across CH 449.90 m started to respond with rising reservoir levels in early June 1991. So did the dam seepage measuring chamber's volume start to increase. The maximum piezometric level reached was to el. 412 m, and the maximum seepage measured was 13 l/s. Seven B series cored holes were drilled along the trace of the access adit Da from the surface because it was suspected that water came into the downstream dam cavity not from across the dam axis grout curtain but via downstream across the access adit Da upstream into the dam cavity. (See Fig. 5) Seven cored holes were drilled to a total of 455 m. The maximum depth drilled was 110 m. A 2 m cavity encountered in B6 was proven to be connected to cavity no. 4 of the dam body. When B6 was drained into the access adit Da it was noticed that seepage in the dam SMC reduced. Later a dye test too proved that the cavity in B6 was directly connected to cavity no. 4, proving also that water came via downstream over the access adit to the dam body. Many as 8 holes in the vicinity of the cavity B6 was later driven into the access adit Da to drain off the water getting into the dam body thus solving the problem. The added leakage was only about 8 l/s.

About 3 km far away in Killekandura ara on the left bank downstream of the dam a leakage of about 40 l/s occurred when the reservoir was around el. 424 m noticed in June 1992 for the first time. Farmers of the area had observed a wet soggy patch developing in the paddy field during the dry spell of the year. The leak in the area for the farmers around Killekandura ara was a bonus, where water was assured right through out the year. How water gets into this area is still not well understood.

14.0 The Burst 22. October 1992

The reservoir level was raised continuously and was on its way to top water level el. 460 m. Most tests done, the power stations machines were ready for generation. The maximum elevation attained by the reservoir was to el. 439.51 on 14th October 1992. The main spring (at M3) increased until the end of July but stayed constant at 77.5 l/s from the beginning of August until 22nd October 1992 though the RWL increased by 8 m (431 to 439). The water of this spring became muddy during the 21st October 1992.

Practically all surface, and right bank cutoff adit piezometers and a few of the dam right abutment cut off piezometers and D5 in the abandoned adit followed the RWL very closely with about one day's time lag (see Fig. 11).

On the 22/10/92 at approximately 13.00 hours a spectacular water burst occurred which turned the lower lying small spring which gave 77 l/s into a torrent of muddy water.

The burst developed at around el. 400 m in upper reaches of the same gully where the first leak developed in June 1991 which is located on the right bank of the Walawe Ganga some 320 m downstream of the toe of the dam. The reservoir water level was at el. 438.99 when the burst occurred. The sudden outflow triggered an earthslip which resulted in increased flows and increased instability until approximately 24.00 hours on 22/10/92 when the outflow stabilized at 7-8 m^3/s with the discharge of apparently clean water from the hill side. The slope had temporarily stabilized but the access road immediately above the slip contained tension cracks and was closed for safety reasons. The slide uncovered an open conduit in 'limestone' associated with faulted ground.

At the same time with the burst all piezometers in area B dropped drastically to el. 419 - 420 m from el. 438.4 m within about 24 hours. The discharge decreased to 5 m^3/s by the 28th & 29th of October while the GWL went down to el. 415 - 416 m. From the 29th October it was noticed that when the discharge reduced from 5 m^3/s to 3 m^3/s within 24 hours to stay steady for a few days that then the GWL rose by 6 to 7 m. This sudden rise in the GWT is attributed to blockage of the seepage path caused by fallen rock or debris reducing the section controlling the flow. It is interesting to note that groundwater responses of piezometers as far as 2450 m apart as in GW1 close to fault zone F1 in Saddle 1 and GW18 in saddle 4 far from the south end of the grout curtain closely follow in the same pattern in response to the reservoir.

Immediately after the burst steps were taken to lower the reservoir water level in a rapid but controlled manner. By 18 hours the low level outlet was fully open and passing about 60 m^3/s. A fault in the low level outlet guard valve necessitated the closure of the valve from 19 hrs. on the 22/10/92 to 12 hrs. on 23/10/92. At the sametime detailed monitoring of the right bank instrumentation was going on.

Throughout 'the burst' incident (see table 8) groundwater levels in the immediate right abutment of the dam, 4 upstream piezometers and 2 downstream piezometers in adit Db have been affected. Two piezometers on the trace of the access adit Da just downstream of the dam indicate also to have been affected. (B Series).

Weir flows on the Kalunaide ara and on the depression opposite Mattihakka ara (saddle 1 area) also have indicated a quick reaction to the burst. It was also noted that the left bank's Killekandura ara leak had reacted to the sudden big leakage.

All these facts confirm the great extent and the perviousness of the semiconfined aquifer on which the reservoir is perched on. Piezometer responses in adit Db, dam right abutment cutoff, which are only about 10 m below invert, i.e. at el. 380m approximately and the dam cavity no. 4's response before the drainage network installed in the access adit Da indicate fingers of area B very comfortably running into area A of the dam itself. (see Fig. 5 and 7).

The evidence obtained indicates that the leakage has opened up a widely spread natural drainage network. The natural drainage, which had already got blocked once issues leakage of 2 cumecs, reading a difference of about 10 m between RWL and GWL so far.

The foundation piezometers of the dam did not react to the leakage and the right bank GWL fall. The piezometric levels in the valley, downstream of the core correspond to approximately 1/3 of the reservoir head. The seepages are very low, less than a l/s as measured in the seepage measuring chamber at the toe of the dam. The deformation of the dam is normal. All these facts confirm that the behavior of the dam foundation is completely satisfactory.

15.0 International Expert Opinion

After the burst international expert opinion was sought. Initially three experts were nominated by the government for what was called third party review panel. However since they couldn't have got together till February 1993, the client desperate to consult them as early as possible permitted one of the experts colleagues to visit site in order to report their joint opinion with relevance to the terms of reference, before Christmas 1992. The site visit was made in late November 92 and discussion with the interested parties were held in early December 92. The following are the terms of reference and the main conclusions and recommendations of the Third Party Review Report SMW-1 submitted in late December 1992 by the two experts jointly:

Terms of Reference for the Review Panel

1. To assess the stability of Samanalawewa Dam, as built and under the presently hydrogeological conditions.
2. To assess the stability of the right bank of Samanalawewa Reservoir under present and short term future conditions before remedial measures are complete.
3. To review available proposals for conceptual design of remedial measures for the Samanalawewa Dam and Reservoir and to suggest on the most appropriate proposal.
4. The Review Panel Report and recommendations should be completed and issued to Ceylon Electricity Board by 25th December, 1992.

Main Conclusions and Recommendations

1. "The grout curtain has no effect on the seepage control in the right bank ridge." This is due to the fact that the curtain is not connected to a reasonably watertight formation, particularly in depth. The extension of the grout curtain so as to make it efficient would be very costly, time-consuming and of doubtful result. It is recommended that no additional work be carried out on the curtain.
2. The main dam safety, as it can be assessed from the results of the ample instrumentation installed both in the embankment and in its foundations, is not jeopardized by the high water table which developed in the right bank after the impounding of the reservoir in June 1992. The pressure release resulting from the water burst of 22 October, 1992, has a global beneficial effect, the present conditions being more favourable than those which prevailed prior to burst. These conditions can be tolerated without risks for some time. However, for the dam safety over the long term it is recommended that the seepage flow in the dam foundations be better controlled, by additional drainage, and means to render the right bank ridge more watertight.
3. The stability of the reservoir right bank under present and short-term future conditions is reasonable. However, the sudden opening of new springs, more or less similar to the water burst of October, cannot be ruled out, owing to the high water table existing in the whole ridge. Such springs could punch the overburden materials covering some of the gullies slopes, particularly in the areas of the southern saddles. But it is considered that such possible events would not threaten the safety of the scheme.
4. Remedial measures are required to ensure the long term stability of the scheme and to reduce the amount of leakage from the full operational level of the reservoir to an acceptable value. Our best guess of the full reservoir leakage in the absence of remedial watertight treatment is between 10 and 20 m^3/s. After a detailed review of the conceptual design prepared by the Consultants, we generally support their findings. The most appropriate remedial measures both technically and economically, consist of additional drainage of the downstream side of the ridge and placement of an impervious blanket on the upstream side where the water ingress zones are located. Some recommendations on these remedial measures are made in the following sections.
5. A drainage gallery should be driven along the right bank abutment of the main dam, extending 300 m to 500 m downstream, so as to "organize" and control the natural (but inadequate) drainage created by the October water burst. This gallery should be equipped with a large number of high capacity drain holes and with a monitoring system consisting mainly of flow gauges and piezometers.
6. A blanket, made of properly graded fine materials, has to be placed on the right bank slope and on the bottom of the reservoir. Placement in water (wet blanketing) is the preferred procedure, for its advantages regarding the programme of the works (particularly no power shut-down) and for its probable higher efficiency as compared with a blanket placed in the dry (spontaneous penetration into small voids). Special investigations are required to design the most efficient procedures and to determine the extent of the area to be lined. We agree with the investigation programme proposed by the Consultants.
7. It will be particularly valuable to know the flow conditions for the low reservoir levels which should be obtained soon, down to El. 424. In addition it is recommended to test the effect of a new rise, up to El. 430, so as to check whether the underground water paths are stable or otherwise. (accurate seepage measurements at least once a day or for every change in the Reservoir Water level not exceeding 0.50 m).
8. The main and most vital reservoir remedial works to be implemented for the benefit of the project is to reduce the leakage to an acceptable value (normally not more than a few percent of the average annual flow), depending on the flow, if any, which should be released downstream of the dam.

For the stability of the right bank ridge, a drainage network, in particular in the vicinity of the right dam abutment, is essential to secure adequate safety on a long term basis, as a second line of defense in case of deterioration of the sealing works provided upstream of the ridge.

Watertightness and drainage measures cannot be considered separately and should be associated in the same remedial works design.

The full panel met in February 1993. Finally the panel consisted of four members and not three members. The colleague of one of the panel members, who visited site was the fourth member. Right bank leakage Review Panel report no. 2 (SMW-2) was the outcome of the full expanded panel. There were notable differences between the SWM-1 and SWM-2.

The following is an **executive summary of the terms of reference and response, the full panel made.**

1. "To assess the stability of Samanalawewa dam, as built and under the present hydrogeological condition"

Response: The dam is a conventional central core rockfill on a hard rock foundation designed and constructed to current standards and constructed of satisfactory materials. Its stability could not be affected by many times the maximum credible leakage that might pass through the right abutment ridge, with full reservoir level of el. 460.

2. "To assess the stability of the Right Bank of Samanalawewa Reservoir under present and short term future conditions before remedial measures are complete".

Response: The right bank ridge is of non-erodible and non-pipeable rock. There are shear zones, and open joints that exist to great depth. The right abutment perimeter of the reservoir will safely pass any amount of leakage for full reservoir before any measures are taken to reduce leakage.

3. "To review available proposals for conceptual design of remedial measures for the Samanalawewa Dam and Reservoir and to suggest the most appropriate proposal".

Response: The extensive grout curtain carried out according to current standard practice has not adequately sealed the abutment even for the low partial filling. The unusual geological conditions of open cracks at great depth make improvements of the curtain impractical. The most appropriate measure, with the probability of significant reduction in leakage, is blanketing.

The following are the **conclusions the full panel made.**

1) Dam - The dam is safe for full reservoir and is independent of right ridge leakage conditions.
2) Right Abutment - The right abutment provides a reliable perimeter for the reservoir at full storage level and any credible amount of leakage. Work is required to reduce leakage for economic, not safety, reasons.
3) Geology - Rock foundations are excellent for the dam. However, tectonic activity has resulted in a sheared, and fractured rock where the fissures are open to a great depth in the right bank ridge. The rock, therefore, is excessively pervious.
4) Right Abutment Grout Gallery - In contrast to the adequate rock conditions on the left bank, the rock on the right bank was found to present adverse conditions as more exposures became visible during construction. For these unusual conditions it was logical to increase the grouting program. The very deep open cracks made the curtain ineffective in adequately reducing leakage.
5) Trial Filling - Combining the knowledge of the very low groundwater levels in the right bank with the high grout takes gave rise to concern for high leakage, and logically prompted the trial filling. The test to water el. 402 m confirmed this concern.
6) October 1992 Event - During the first filling the sudden release created only a local stability problem in the overburden material. The stabilised 2 m^3/s leakage for water surface 426 is high and suggests that above 10 m^3/s for full reservoir is possible. This event verified the need for measures to reduce leakage.
7) Measures for Reducing Leakage - Evaluation of geology data, experience, and investigations indicate that wet blanketing is the most promising means of reducing leakage.
8) General Abutment Drainage - The reservoir abutment rock drains itself freely and safely. With reservoir level higher than el. 440 some overburden slides can be expected. They would be of no consequence and no ridge drainage is necessary.
9) Dam Right Abutment Drainage - The dam and the right abutment rock just downstream of the dam, is safe against high abutment piezometric levels and drainage is not necessary. However, there is an unlined access gallery in exactly the correct location. Additional controlled drains in the gallery are advisable in this transition zone between dam and abutment.
10) Outlet Works - Since it is firmly judged that the right abutment and dam are safe for full reservoir, no modification to the outlet works is necessary.
11) Flood Season 1993 There is no risk to the dam or right bank stability should the reservoir fill before remedial works are carried out. Frequent measurements and surface observations should be made during the rising reservoir.

12) Monitoring during Blanket Placement - The principal observations will be measurement of the leakage and observance of sediment and coloration of the leakage, and recording of piezometric levels. At the bench end where placement takes place, the volume discharged for a meter progress in extension of bench should be recorded. Soundings would be made.

13) First Filling - There should be no established short or long term objectives of permissible piezometric levels or of limits of permissible leakages.

Further to the conclusions comments were made on different aspects. The following are some of these comments.

Blanket - For the conditions at Samanalawewa the wet method is the practical and economical method to seal the river bed and the lower slope of the right ridge face. The water level would be the level which enables power plant operation during placement. Above that elevation, dry placement is appropriate to thicken the existing thickness of the saprolite blanket, were it considered necessary. Little dry placement is anticipated to be required.

The mechanism of sealing by the wet method for the open jointed hard rock, where it is not covered by an adequate thickness of saprolite, is that leakage into the rock mass will draw the fine material into fissures in the rock mass until it is sealed.

Strength of underwater placed material is not needed. The important consideration is to use fine material such as the saprolite. Some sand sizes are of course desirable. The important material is minus # 200 mesh. Occasional rocks, but not a load of rocky material, are acceptable.

Drainage - During the wet season before the blanket has been in place, additional drainage of a provisional nature should be provided. The existing access galley D(a) in the appropriate location for this drains. All drains should be fitted with a closure valve. The drainage adit suggested in Report No. 1 is no longer considered necessary, because new local slides in the overburden can be accepted (see figs 12 and 13).

Outlet Works - A greater capacity low level outlet could have been useful in the light of events. However at this stage in the project it is not justified to increase capacity for implementation of the blanket, since it is firmly judged that the perimeter of the reservoir and dam are safe for full reservoir.

Flood Season - The panel reasserts its position that there is no risk to safe performance of the dam and right bank for full reservoir operation before the blanket is constructed. However, as a principle in dam engineering, spillway gates should remain open when remedial works are judged to be necessary.

Reservoir Filling - .... to recommend a "long term allowable leakage amount" and a "long term allowable underground potential head". The answer is that no such criteria are advisable as they may conflict with the proper future actions, dictated by sound engineering and economic requirements.

Samanalawewa leakage unprecedented.

In a separate page - captioned, Some cases of sealing material placed underwater, 14 cases of sealing leakage or constructing blankets underwater were submitted with the report SMW-2.

Number 15 was a comment on Samanalawewa which is attributed as an answer to a question raised during the discussions. It stated that at this time a precedent sealing of a pervious ridge by underwater dumping of impervious material, is not recalled. However, it should be more simple and effective than other precedents of underwater sealing. A narrow abutment ridge is usually handled by drainage galleries and a dry placed blanket over a filter on the upstream face of ridge (Yacambu, Venezuela 182 m height). The Samanalawewa ridge is very wide and stable: quite different case than Yacambu.

Since 22nd October 1992 (the burst occurred) reservoir levels have been lowered and maintained at or below el. 430 m. Groundwater levels have remained some 10 m below reservoir level and the leakage volume has continued at approximately 2 m^3/sec. The spillway gates have been kept open permanently (the sill level 446.7 m) taking maximum precaution to ease a major flood. Apart from the eight drainage holes (DR1 to DR8, adding to 203 m of drilling) done earlier in the neighbourhood of boreholes B6 and B7 from the soffit upwards through the cavernous limestone to the weathered overburden from the downstream access adit Da, to intercept the water coming into the dam cavity via downstream, 21 more drainage holes (DR 9 to DR 29, adding up to 980 m of drilling) have been done right along the access adit Da, to the dam cutoff point, after the panels' recommendation. (See Fig. 13).

16.0 Project Consultants Approach

As indicated by the Review Panel remedial measures to reduce leakage at Samanalawewa are for economic reasons, not safety. Both the project consultants and the review panel have indicated that.

i) there is no possibility of the dam or the right bank failing and therefore there is no risk of a catastrophic disaster.

ii) if the groundwater level in the right bank ridge exceeds that at which the water burst occurred on 22 October 1992 at some time in the future then similar bursts in other area are likely.

iii) additional water bursts, while being alarming when they occur, will not compromise the safety of the dam and reservoir; however they are likely to result in increased leakage losses and this would not be acceptable on economic grounds.

Since the burst, measures are taken to ensure that the groundwater level in the right bank is maintained well below the level at which the water burst occurred (El.438 m) so that the risk of further instability and hence increased leakage is minimized. Without increasing the current water losses from the reservoir and to achieve groundwater control the project consultants have discussed of two approaches which possibly could be adopted. These two approaches are as follows.

(i) Ensure that reservoir levels never reach the level at which the groundwater caused the water burst. This could be achieved by operating the projects as run-of-river scheme provided additional spillway facilities (called Second Spillway) are constructed to ensure that reservoir levels remain below El.438 m during floods.

(ii) Create a positive cut-off on the right margin of the reservoir by either :

a) Extending the existing grout curtain, which has been agreed by all parties to be technically impractical or

b) Constructing an upstream blanket over the areas of reservoir water ingress to control leakage, (and hence groundwater levels,) to less than the downstream requirements at Full Supply Level. The blanket could be constructed either in the dry, using conventional techniques, or in the wet by tipping or dumping.

In the case of provision of a Second spillway to pass the design flood of 3600 m^3/s a tunnel type spillway to be located below the existing spillway with its forebay crest elevation of approximately El. 430 without gate can be considered to control the reservoir water level for the Run-of-the-River operation use.

The Consultants have further said that it is to also be noted the fact that the run-of-the-river type operation will result in the significant decrease of "Firm Energy".

According to the results of optimization study of Samanalawewa reservoir in the "Additional Study" carried out by Electrowatt Engineering Services in 1985 and 1986, the energy production at the various maximum water level (low water level is fixed to be El.424 m) are summarized as follows:

Max. Reservoir Level (m a.s.l.)	Firm Energy (GWh)	Secondary (GWh)	Total Energy (GWh)	
460	366.47	58.83	425.30	<= Storage type operation mode (Blanketing)
455	338.94	83.15	422.09	
450	297.28	120.14	417.42	
445	258.74	153.34	412.08	
440	206.63	200.12	406.75	
435	147.76	253.27	401.03	
430	98.60	296.49	395.08	<= Run-of-River operation mode (Second Spillway)

As seen from the above table, if the Second Spillway is adopted for the permanent solution, though the total energy (firm plus secondary) will be reduced by approximately 30 GWh compared with the original scheme the firm energy itself will be decreased to 27% of the original scheme, 120 MW.

Taking into consideration the construction costs including all the previous costs spent for construction of dam, waterway and powerhouse, etc., and their expected annual energy production, economic and financial evaluation of the two alternatives were carried out by consultants and the Economic Internal Rate of Return (E.I.R.R.) and Financial Internal Rate of Return (F.I.R.R.) of each scheme are calculated as follows:

	E.I.R.R.	F.I.R.R.
A. Run-of-the-River type operation mode with provision of Second Spillway	3.2%	7.1%
B. Storage type operation mode with provision of Blanketing	14.0%	7.6%

From the above, it is very clear that the storage type operation mode with provision of blanketing is economically much superior to the run-of-the-river operation mode with provision of second spillway. Furthermore, all the parties concerned have agreed that the best approach to control the groundwater level, and hence leakage, in the right bank of Samanalawewa is to use the wet blanket approach. This approach, carried out when the reservoir level is above minimum operating level, ensure the minimum disruption to power generation (the scheme will be running as a run-of-the-river plant for the construction period).

The ultimate aim of the proposed remedial measures (wet blanketing) is to ensure that the Samanalawewa Reservoir provides the storage for power production as originally anticipated.

The river bed between 700 m and 1700 m upstream of the dam on the Walawe is assumed to be the area where significant water ingress to the right bank is occurring and is to be the main target for blanketing. The approximate fill volume for this section is estimated to be 500,000 m^3. This conclusion is supported mainly by the following observations.

1) Before impounding, the groundwater level recorded in the right bank was flat at about el. 380 m and fluctuated in response to changing river levels. The river bed is at el. 380 m around the centre of the target section between 700 m and 1700 m upstream of the dam.
2) Three major faults F-1, F-2 and F-3 intersect in the target section of the reservoir (and so also is GRA4 and CHA2).

Apart from the main zone of water ingress which is likely to be within the main target section it is possible that other zones of ingress may exist. Therefore provision must be made for additional blanketing work to be undertaken once the main target section has been covered. The extent of this follow up work is said to be unknown.

Three alternative methods of constructing an impermeable blanket underwater have been identified and studied :

1. Dumping using dredgers
2. Dumping using bottom dump barges
3. Side dumping using dump trucks

To assess the technical merits of each approach, similar aspects of construction have been assumed. The assessment have shown that dumping using bottom dump barges has the most advantages over the other approaches, while dumping using a dredger has the most disadvantages. Side dumping is almost as advantageous as using the dump barges except a few aspects, particularly flexibility.

17.0 The Bureau's View Expressed

The "Bureau" (Central Engineering Consultancy Bureau, Colombo 7), as an independent body, for the first time stepped in to Samanalawewa after the first trial impounding in August 1991.

Inspecting the site in May after the second trial impounding in March 1992 the "Bureau" expressed serious apprehensions about the response of the right bank hill with the rising reservoir. The apprehensions were based in field observations in respect of

(1) the quick potentiometric response of the piezometers upstream and downstream of the grout curtain
(2) the discharge of water from the grouting gallery piezometers when opened
(3) the development of wet patches and springs rising higher up in elevation as the reservoir rose as seen in the area downstream from the toe of the dam on the right bank and up the Kalunaide ara, and
(4) the state of the geology and the hydrogeology of the immediate right abutment and right bank saddle area.

Migration of water under pressure to the sensitive areas will cause high uplift and pore pressures in the rock and overburden. The over burden will not be able to sustain such a situation. A calculated risk can be taken and fill the reservoir to the minimum operating level el. 424 m and thus be in a position to test and commission the generating units. Impounding upto el. 424 m may cause some embarrassing leakages and slides in the overburden but no unsustainable instability problems with the rock mass as a whole are presently foreseen. Raising of the reservoir beyond el. 424 m need be eschewed and the power house operated on a run-of-the-river basis until suitable measures are taken to reasonably ensure that migration of water under pressure to sensitive areas and zones is prevented.

Such measures can be

(1) to stop the entry of water to the hill mass by providing an impermeable blanket in the river bed and along the flanks
(2) reinforce the grout curtain both in depth and length and where necessary in thickness so as to provide and effective cutoff and
(3) provide internal drainage downstream of the existing grout curtain to arrest the leakage.

The first alternative is not practicable at this stage and is in any case only conceptual and uncertain because the source of entry of water are not known and may be wide-spread.

The second alternative is achievable but the extent and cost for getting the desired result could be unacceptably high. Under the circumstances the only manageable solution appears to provide internal drainage which will be effective and cheap solution. If leakage is excessive, it could be pumped back into the reservoir thus conserving water but losing a small percentage of energy.

Some field trials will be necessary before working out the details of the drainage arrangement. This can be conveniently done from the existing grouting gallery. The monitoring of the surface piezometers response keeping all the internal cutoff adits and abandoned adits piezometers open to drain was proposed to be done by the 'Bureau' but done only after the burst.

The 'Bureau' in February 1993 presented to the review panel its proposal 'Remedial measures for leakage along right abutment hill'. The report stated the following that:

* It pertains primarily to the immediate right abutment hill of the dam and does not include the problems of the reservoir tightness in their totality, which will consist of, but not limited to, the two major saddles further south and the likelihood of the main reservoir at full supply level feeding the adjoining aquifer met with while tunnelling for the headrace.
* There is need to device measures to reduce the development of pore pressures for stability of hill which shoulders the dam and to reduce the leakage from the reservoir reaching the critical zones and in the process to minimise the total leakage losses to the extent possible.
* Internal drainage in multiple tiers has been suggested to reduce the pore pressure in stage and thus reduce the flow velocities in the fissures as also to make spot control of leakage more manageable. A grout curtain sub-parallel to the flow of the river surrounding the internal drainage system has been proposed to reduce reservoir leakage finding way to critical zones (see Fig. 14).
* In the unlikely event of the leakage being still beyond acceptable economic limits, pumping back to the reservoir will be provided against a head of upto 80 m to enable its utilization for generation of power with the available head of 325 m.
* Augmentation of the present disproportionately low bottom outlet capacity has been suggested not only for safety of works during initial filling and for safety and regulation requirements during implementation of remedial works but also on a permanent basis for a sensitive reservoir like Samanalawewa where problems related to reservoir water-tightness are distinctly existent.
* The implementation of remedial works will be a time-consuming steady process which may run over a period of upto 4 years. Interim power generation arrangements have therefore been suggested for partial benefits to the extent of about 40% or so.

The Bureau's comments on report No. 2 (SWM-2) of the Review Panel were as follows.

The Bureau stated that it respect the conclusions and suggestions of the Panel but considered it their bounden duty to apprise the Government of their views on the full implementations of the conclusions of the Panel before the Government embarks upon their implementation.

The Bureau's comments and views itemwise are discussed in the succeeding paragraphs.

The safety of the dam structure as a whole was confirmed by the Bureau as early in August before the burst. The structural reliability of the rock along the right abutment ridge against any breach to take place was never in doubt. It's primary concern was about the instability of the mantle of weathered rock and overburden.

Commenting on wet blanketing for reduction of leakage, the Bureau gave the following reasons as to why it gave up it's own consideration for clay blanketing.

(1) River valley is narrow and steep and hence unsuited for a stable blanket.
(2) There are no well defined points from where the leaks could be taking place. Blanketing would thus be a hit-and-miss exercise.
(3) The leakages could be emanating from limestone bands, faults, and fissures along either bank of the river and not necessarily from the right flank alone.
(4) Such vulnerability is also expected to exist along the left flank of Walawe and both flanks of Belihuloya as well, as indicated from the geomorphological and structural delineations of the geological features and discontinuities.
(5) The aquifer under question is very extensive. It does exist even at 3 km downstream on the left flank of the dam in addition to that on the right bank of Walawe. This was evident from the response of leakage along the left flank of Walawe Ganga about 3 km downstream of the dam when the blow-out took place in October 1992.
(6) General review of the surface geological features indicates that the vulnerable areas of leakage along the right bank of the reservoir along Walawe extend upto at least 2 kilometers from the dam or may be upto almost the power intake.
(7) A doubt exists that the location upstream of the dam, identified by the Panel as the most probable major ingress point could even be an outlet point of the aquifer. This needs more study and investigation.

Commenting on the technique proposed by the Panel to seal the leak by underwater clay dumping, the Bureau elaborating the nature of the confined aquifer on which the reservoir is perched stated that there is no dependable traction force to suck in the dumped material from the reservoir for efficient, effective and fast caulking of the passages from where the ingress takes place. The proposal to seal the ingress passage(s) by blanketing in the manner as proposed by the Panel may not therefore be as dependable and effective as it is being considered to be. Apprehensions such as these may have discouraged the use of such technique to seal a steep and pervious ridge holding a confined aquifer in the past and no known precedent therefore exists. However, if undertaken, it will be a trial for the first time. It is important to note here that the present piezometric head difference is depandent entirely on unorganised natural burst flow & has in the past reduced for 5 cumecs to about 2 cumecs and could be blocked any time by internal rock falls.

During discussions with the Bureau on February 18, 1992 a member of the Panel on being asked the question, mentioned that no estimate is presently possible of the ultimate volume of dumping required for a successful result. Presently it can only be said that it could be many times more than what can be estimated at present. The cost and time aspects of the proposal thus remain loose.

The Panel of experts have given an unreserved view about stability of dam & reservoir with FRL at el. 460 m. Their only apprehension is loss of water by increased leakage. However, such a situation has not been allowed to occur for reasons not known to the Bureau.

Commenting on the stability of the immediate right abutment in particular the Bureau stated that:

It is true that the ridge as such is safe against any breach causing destruction of the reservoir. But the likelihood and possibility of blowouts and severe landslides is foreseen which in turn would adversely effect the stability of the dam itself in the extreme right abutment area apart from unsustainable loss of water. Geomorphological, geological and geo-hydrological features along this ridge suggest that the zone along the fault immediately downstream of the dam is the most susceptible of all areas in the right abutment ridge for blow-outs and slides. The CAL zone just skirts the dam along its right abutment. This consists of charnokitic gneiss with 30% impure-crystalline limestone which has caverns associated with faults (F1 type) and will get charged with leakage water as the reservoir rises, causing uplift and bursting pressures on the thin mantle of overburden material resulting in progressive erosional slides and blow-outs in the close vicinity of the dam on the downstream side. Displacement of underlying erodible materials would, in turn, progressively dislodge the overburden above it in the long run. This can undermine the area on which the dam abutment is resting which can be very serious. (see fig. 15).

18.0 Conclusions

Reservoir impounding was originally scheduled to begin in April 1991. The first trial reservoir impounding took place in June 1991. The reservoir bearly rising upto one third of its designed height started to leak from 320 m down stream of the dam proving how seriously pervious the right bank ridge is. The second trial impounding was in March 1992 after the massive right bank cutoff 1300 m x 100 m grout curtain, where 13500 tons of cement was injected to 53000 m of grout holes. This cut-off measure which cost half the dam and spillway, never created a hydraulic gradient across it. The right bank groundwater table even with the massive grout curtain kept on rising with the reservoir level. The October 1992 burst, with a steady leakage around 2 cumecs (for reservoir levels from 426 m to 433 m) has kept the ground water level almost 10 m below the reservoir level thus improving the stability of the right bank area. The natural drainage did get block once reducing leakage, it could get blocked again. Blow-outs could happen once again causing more leakage and erosion.

The most vulnerable and the most dangerous area susceptible to failure and erosion is just downstream on the right bank of the toe of the dam in the Mattihakka fault F1 area which in not far from the reservoir across the right abutment. The final 20 m of the dam abuts highly weathered rock which consist of an erodible complete weathered seam which took the bulk of the cement grout of the right abutment. In general rock levels just downstream of the dam on the right bank is around 420 m. The ground level at the toe of the dam in around 370 m. Suitable drainage arrangements in this area was proposed for safety reasons not taking any chance.

The history of this project, in detail shows how the problem of reservoir leakage was proposed to be confirmed before tender. However the detailed site investigations were done only during construction. By then the dam contractor was two years into the construction programme.

The reservoir is perched on a very previous extensive aquifer. Clay blanketing which would cost another nearly half the dam & the spillway is unaffordable because it is not guaranteed. If Samanalawewa stand to be only a leak problem, why the reservoir is not filled to FSL (460 m) but kept only at 430 m, using it as a runoff the river project, demand a valid explanation.

Acknowledgement

The author wishes to thank the Central Engineering Consultancy Bureau for permission to publish this article for the benefit of the Engineering Community of Sri Lanka. Dr. A.N.S. Kulasinghe, Chairman, CECB's tremendous experience and insight used in connection with Samanalawewa is very much appreciated. Mr. G.G. Jayawardhana, General Manager, CECB is thanked for his encouragement. Mr. G.N. Tandon, Chief Engineering Advisor with CECB is thanked for his suggestions and discussions.

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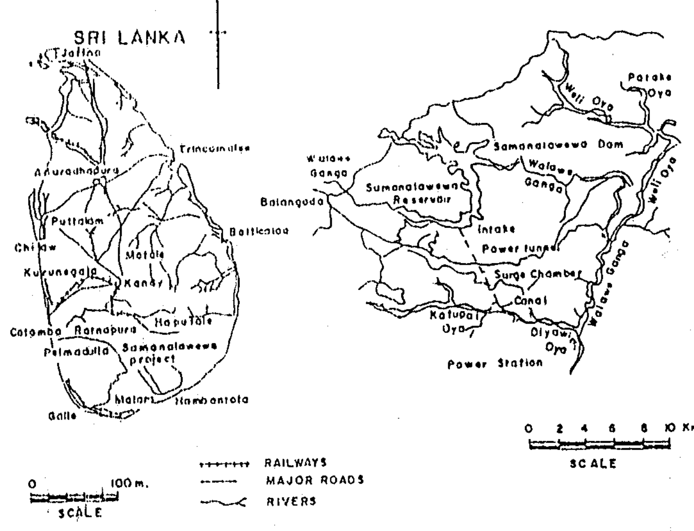
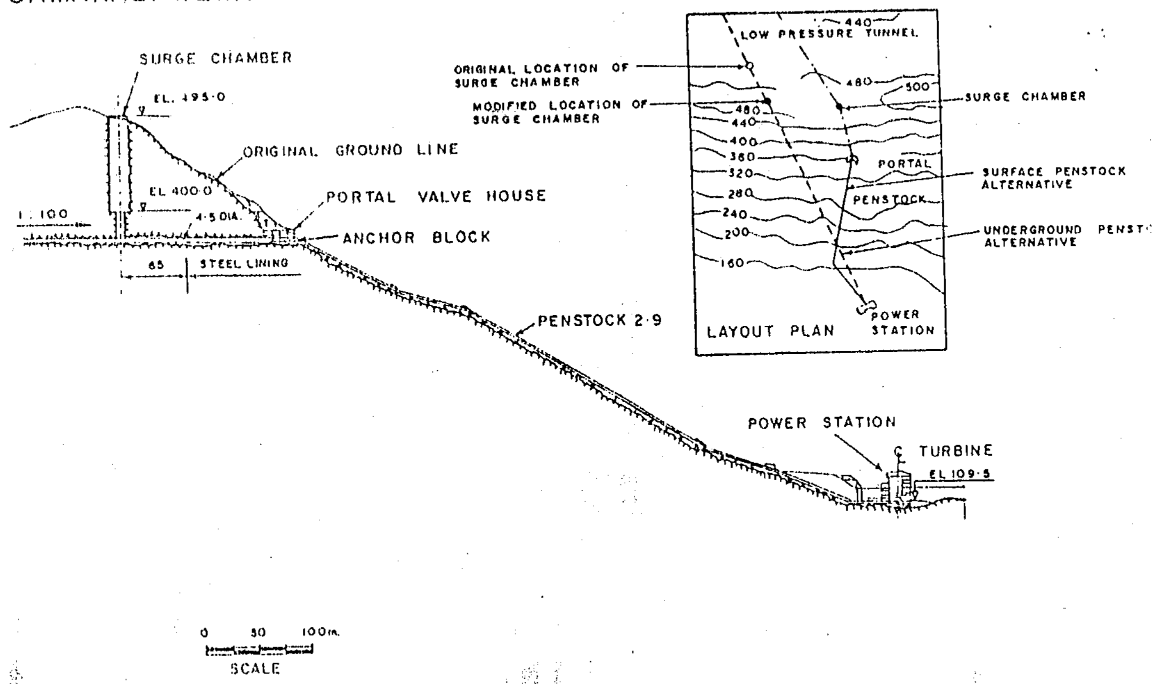
* Unpublished

Table 1
Samanalawewa Hydro-Electric Project - Principal Feature

1.	Samanalawewa reservoir	
	Catchment area	341.7 km²
	Annual average runoff	$598 * 10^6$ m³
	Flood level	460.7 m
	Normal high water level	460.0 m
	Minimum operating level	424.0
	Live storage above minimum operating level	$254 * 10^6$ m³
2.	Samanalawewa Dam	
	Type	Rockfill dam with clay core
	Crest length	530 m
	Dam height	100 m
	Volume of fill	$4.48 * 10^6$ m³
3.	Spillway	
	Type	Gated overflow
	Capacity	3600 m³/s
	Number of gates	3
	Size of gates	12 m wide * 14 m high
	Sill level	446.7 m
4.	Diversion tunnel/low level outlet	
	Number of diversion tunnels	2
	Diameter	6.8 m
	Lengths	482.2 m, 501.6 m
	Diversion capacity	1450 m³/s (100 year flood coffer dam at el.403m, 35m in height)
	Low level outlet	Jet flow gated, emergency 2m*2m Jet flow gated, irrigation 0.6m dia. With one diversion tunnel dam elevation at 429m for a 100 year flood.
5.	Power supply tunnel system	
5.1	Intake	
	Type of gate	Fixed wheel type
	Size of gate	4.5m high * 3.5 m wide
5.2	Low pressure concrete lined tunnel	
	Diameter	4.5 m
	Length	5423 m
	Intake invert level	416.75 m
5.3	Surge chamber	
	Type	Restricted orifice type
	Diameter of riser	6 m
	Diameter of chamber	18 m
	Height of chamber	94 m
5.4	Penstock	
	Diameter	3.85 m, 2.85 m
	Length	840 m
5.5	Valve house	
	Valve type	Butterfly
	Valve diameter	3.85 m
6.	Power Station	
	Type	Surface
	Number of units	2
	Type of turbine, elevation	Francis, 109 m
	Installed capacity of each unit	60 MW
	Speed	500 rpm
	Rated head of turbine	320 m
	Rated discharge	42 m³/s
	Generator transformer	2,75 MVA, 3 phase
	Average energy	462 GWh
	Firm energy	420 GWh
7.	Tailrace canal	
	Type	Open canal
	Length	540 m, to Katupath Oya



SAMANALAWEWA PROJECT LOCATION MAP



LONGITUDINAL SECTION OF POWER INTAKE ARRANGEMENT OF SURFACE PENTOCK

Table 2 - Summary of Exploratory Works Dam/Saddle

Investigations	Total no. of cored holes	Total Length (m)	Maximum Depth of Hole	Exploratory Adits No. Size and Location	Remarks
	Dam/Saddles				
1958 - 1966 ECI Feasibility	45/10	1710/505	84/72	1/18m tunnel 28m, cut RB	-
1972 - 1973 SMEC Feasibility	3/1	184/62	77/62	-	-
1975 - 1978 TPE Feasibility	55/4	2663/549	200/150	2, RB 2, LB	Geophysical surveys, resistivity soundings 3.1 km, seismic 2.24 km stated
1984 BB/GIBB Pre-tender Pre-design	None	None	None	None	Proposed exploratory, dam/saddles
1985 EWI (Proposed) Pre-tender/Pre-design	13/10	1000/1150	120/150	-	Proposed
1986 NK (Proposed) Pre-tender/Pre-design	25/5	1560/520	250/150	-	Proposed Only investigations for dam done
1986 - 1987 NK Pre-tender	35/0	2164/0	170/0	1/100 m RB	Seismic refraction 1320/440
1988 August 1990 November NK/GIBB During Construction	*11/17 *Right abutment/saddles	*1562/3055	*202/230	Exploratory adits(grouting adits) right bank cutoff 2474m access adits 744m	Seismic reflextion 1640 m Refraction 3700 m

ECI Engineering Consultants Incorporated, USA
SMEC Snowy Mountains Engineering Corporation, Australia
TPE Technopromexport, Moscow, USSR
NK Nippon Koei, Tokyo, Japan
GIBB Sir Alexander Gibb and Parters, Reading, London, England

1562/3055, dam/saddles data
RIGHT BANK CUTOFF adits in meters
Access adits E 280.40; D 214.59; Da 249.23
Grouting Adits A 99.60 Ab 103.50
B 132.05
C 211.44
D 312.00 Db 300.00 IC 1315.15
Adit length in meters

1958 - 1966 ECI	Investigated saddles 1 and 2 for natural spillway - poor quality rock incountered.
1972 - 1973 SMEC	Recommended piezometric studies at dam site and saddles.
1975 - 1978 TPE	Identified seepage losses from dam foundation pass its right wing and in Saddle 2. Recommended to establish a network of piezometers in the depression on the right shore (downstream of the dam).
1984 BB/GIBB	Identified that additional exploratory work required to determine precise excavation depth for dam. Further investigations essential to ascertain the stability and permeability of these potentially weak sections (saddles) of reservoir to provide a sufficient reliable estimate of seepage and to provide data for design of any cutoff works.
1985 SECB	In the right bank it is more likely that some under drainage takes place. Seepage loses are likely to occur in the two right bank saddle and the left bank saddle.
1985 EWI	Agreed with what BB/GIBB identified
1985 (additional) EWI	Proposed 1000 m of drilling investigation for the dam foundation, 1150 m of drilling investigations in the Saddles.
1986 NK	Proposed drilling investigations on dam right abutment and saddles with piezometric studies. Said watertable is irregular in shape at the right bank (abutment) and that the groundwater table in the ridge in not known.
1987 JVS (NK)	No proposed drilling investigations on the saddles were done - no piezometric study was done. The report concluded that it is unlikely that saddles might provide any substantial leakage paths.
1988 - 1990 JVS/GIBB	During construction 4500 m of deep drilling investigations with piezometers installed were done on the dam right abutment and saddles.

Table 3 - Samanalawewa Hydro-Electric Project
Project Organization

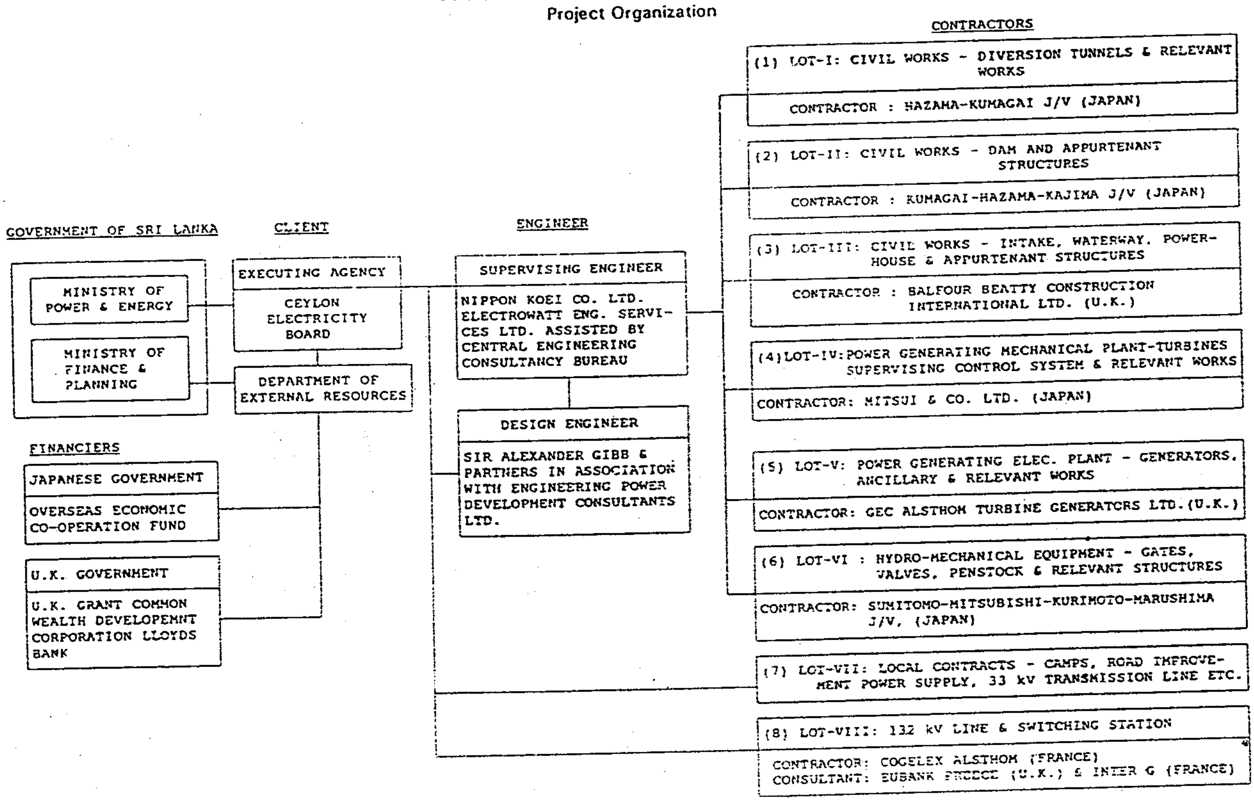


Table 4 - Boreholes proposed/done in Dam, Quarry and Saddles by NK 86/87

Number of boreholes proposed or done	Location	Depth (m) Maximum Drilled	Total (m) Drilling	Remarks
1 (P)	Left Bank Saddle	70	70	Not done
4 (P)	Right Bank Saddles	150	450	Not done
4 (P) 4 (D)	Quarry Quarry	100 120	300 300	All four proposed were done depth maximum not 100 m but 120 m
25(P)	Dam 10 (P) on the Left Bank 15 (P) on the Right Bank including river bed	80 250	320 1240	All proposed was done and even more to deviate the dam axie on the right abutment.
35(D)	10 (D) Left Bank 25 (D) Right Bank	80 170	370 1794	
35 (D) 34 (P)		170(D) 250(P)	2464(D) 2380(P)	

(P) = Proposed
(D) = done

- Right bank core drilling quantity increased to investigate for an advantageous change in the dam axis with reference to depth of weathered excavation.
- Saddle investigations with drilling and piezometer installation were completely dropped.
- Right abutment deep drilling investigations to 250 m (J12) was abandoned at 145.35 m, drill rods got stuck (no piezometer was installed although the groundwater table was dropping).
- J 38 was done to a depth of 168.20 m which also got stuck. This hole was cased to a 110 m. (No piezometer was installed although the groundwater table was dropping).
- The following borehole after J38 was J39, this was done only to a depth of 70.50 m.
- Partial water pressure measurements were proposed to be done in J10, J12, J19 and J26 to study the hydraulic gradient on the right bank. This study as proposed was not conducted.

Table 5 - The Broad Metamorphic Stratigraphy of the Dam and the Saddle Dams

	TPE(Russian) 1978 Classification		JVS/GIBB 1989 Classification	
	Unit Designation	Rock Types	Unit Designation	Rock Types
SADDLES	-	-	GRA5	Predominantly garnetiferous granulitic gneiss
			CHA2	Predominantly charnockitic with 'Limestone'
			GRA4	Interfoliated granulitic & charnockitic gneiss with thin 'Limestone'
DAM			GRA3	Granulitic gneiss 'impervious membrane'
	Upper Granulite bench	Inter-banded granulites, garnetiferous & biotite gneisses 160 - 170 m	GRA2	Interfoliated granulite and charnockitic gneiss with thin 'Limestone'
	Upper Carbon bearing bench	Interbanded biotite and garnetiferous gneiss charnockites and 'limestone' 30% 40-70 m	CAL	Interfoliated charnockitic gneiss and 'Limestone'
	Lower granulite bench	Granulite 20-25 m	GRA1	Garnetiferous granulite (marker band)
	Lower carbon bearing bench	Inter banded 'limestone' charnockites biotite and garnetiferous gneisses 150 m	CHA1	Predominantly charnockite with 'Limestone', interfoliated granulitic and charnockitic gneisses

Table-6 - Sub-division of the immediate dam right abutment an Saddle areas

Area A	Area B
Calcite present in impure crystalline limestone	Calcite absent (as a mineral not encountered)
High groundwater table around el. 419 m	Flat groundwater table around el. 380 m responding to river level changes
Tight ground at depth	Very pervious ground at depth
Weathering limited to calcareous units and faults	Weathering very deep
Influence of hydrothermal alteration limited only in fingers of the area B type is present.	Highly influenced by hydrothermal alteration with many corrosive features and dark green alteration in rock cores.
Some anhydrite CaSO4 present	Smectite, sericite and anhydrite observed in cores - X-ray analysis
Solution cavities rarely found at depth	Discontinuities open cavities found at depths
	Micro breccia observed in thin Sections of Rock

Table 7 - Drilling and Grouting, July 1991 quantities, against March 1992 final

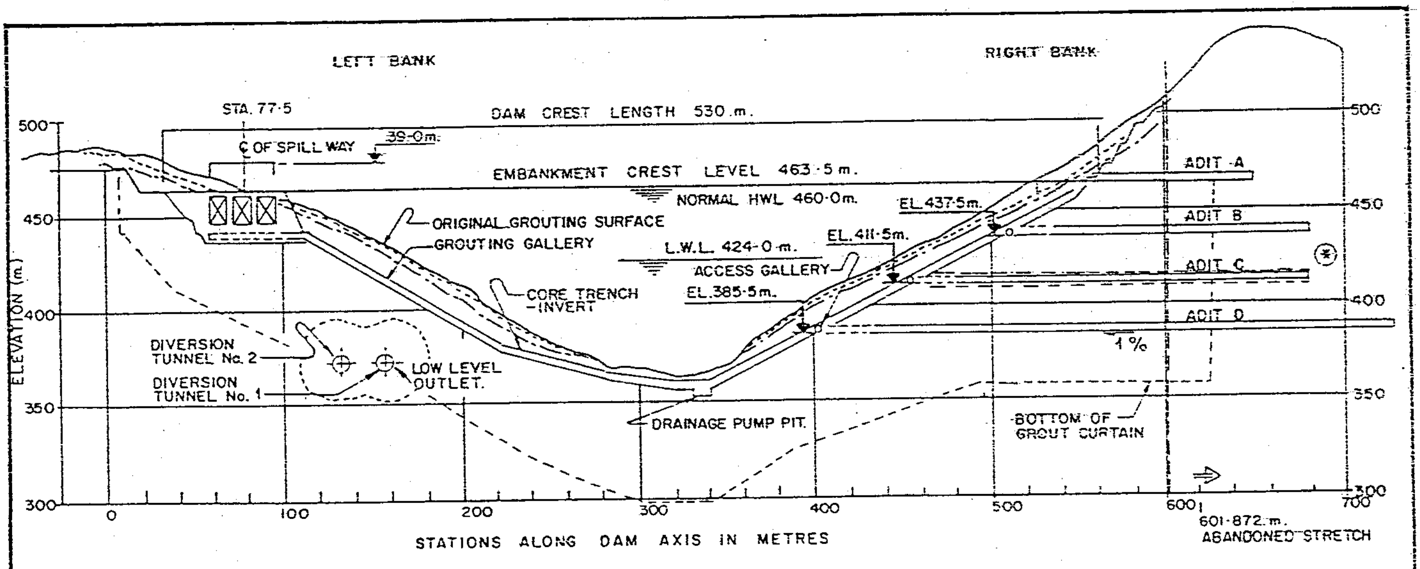
	From Adits Ab & Db, dam right abutment cutoff	From adits I, H & F right bank cutoff adit
Drilling (m)	11,758/15,649	15,061/53,293
Grouting (tons)	982/1254	7035/13,641

Table 8 - Burst 22 October 1992 and its effects

	Location	Response
Piezometers SP59 SP63, SP65, SP67 Sp64, SP68	Dam right abutment cutoff adit DS Upstream Downstream	 Reacted Reacted quickly Reacted quickly
Piezometers RBS1 RBS2,3,5,7,8,9,11, 12,14,15,17 RBS 19,20,21,22,24 & 26 RBS 27,28,29,30 RBS 4,6,10,13,18, 23 & 25	Right bank cutoff grouting adit Adit I - CH 1110 m Adit H - CH 280-985 m Adit F - CH 0 to 280 m Adit E - Access adit Deep piezometer 180 m below cutoff adit	 Reacted quickly Reacted quickly Reacted quickly Reacted quickly Reacted quickly
Piezometers SP 55 (D1) SP 56 (D2) SP 58 (D5)	Abandoned Adit D	Reacted quickly
Piezometers B1, B2, B3 & B4 MS2, B5	Access adit Da along trace from the surface	 Not reacted Reacted somewhat
Piezometers RI6, 7, 8 RI9	Slip area 350 m on right bank from the toe of the dam	 Not reacted Reacted somewhat
GW1,2,3,4,5,6,8 & 13 MG 1,3 & 4 GW 7, 14 & 15 GW 16 (not quite in order) GW 18	Right Bank Saddle 1 area Right Bank Saddle 2 Right Bank Saddle 3 area Right Bank saddle 4 area	Reacted quickly Reacted quickly - Reacted quickly
Weir 9 S1 and S4 M1 DW2	 Kalunaide ara On the depression opposite the Mattihakka ara Total flow from right bank cutoff works	 Reacted quickly Reacted quick Reacted quick
Stream	Killekandura ara 3 km down-stream of left bank	Suppose to have reacted

Table 9 - Grouting Drilling/Grount take, of Dam, Right Abutment and Right Bank Cutoff

	Dam		Dam Right Abutment		Right Bank Cut-off				
Location	Spillway	Dam Body	Adit A,B,C & D	Upstream Adits Ab & Db	Adit I	Adit H		Adit F	Check Holes
Chainage (m)	0-116	116-396	396-612	0 - 100 0 - 300	1315-960	960-620	620-290	290-0	1315-0
	0 - 612				1315 - 0				
Drilling (m)	1974	5548	8018	15,449	3477	18564	9460	20238	1555
	15540				53293				
Grout Take (tons)	7.7	1.9	61.3	1254	307.7	4936.4	1524.6	6748.5	123.4
	70.9				13640.5				
Average (kg/m)	4	0.3	7.6	80	89	266	161	334	79.3
	4.6				256				



LONGITUDINAL SECTION SHOWING DIVERSION TUNNELS GROUTING GALLERY AND RIGHT BANK CUTOFF ADITS

(*) DRAINAGE ADIT 20 m DOWNSTREAM OF DAM AXIS INITIAL PROPOSAL.

SAMANALAWEWA PROJECT LAYOUT

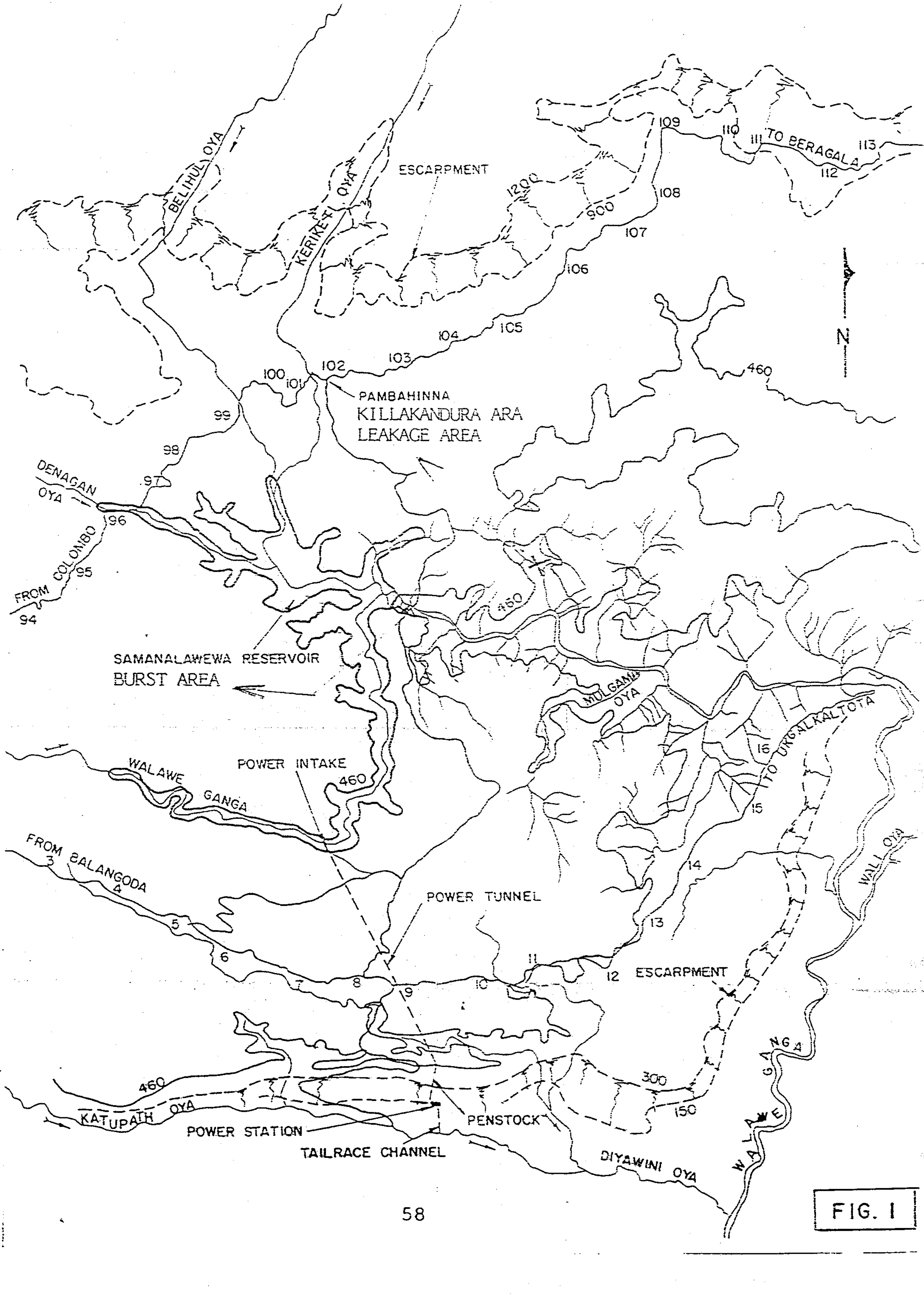
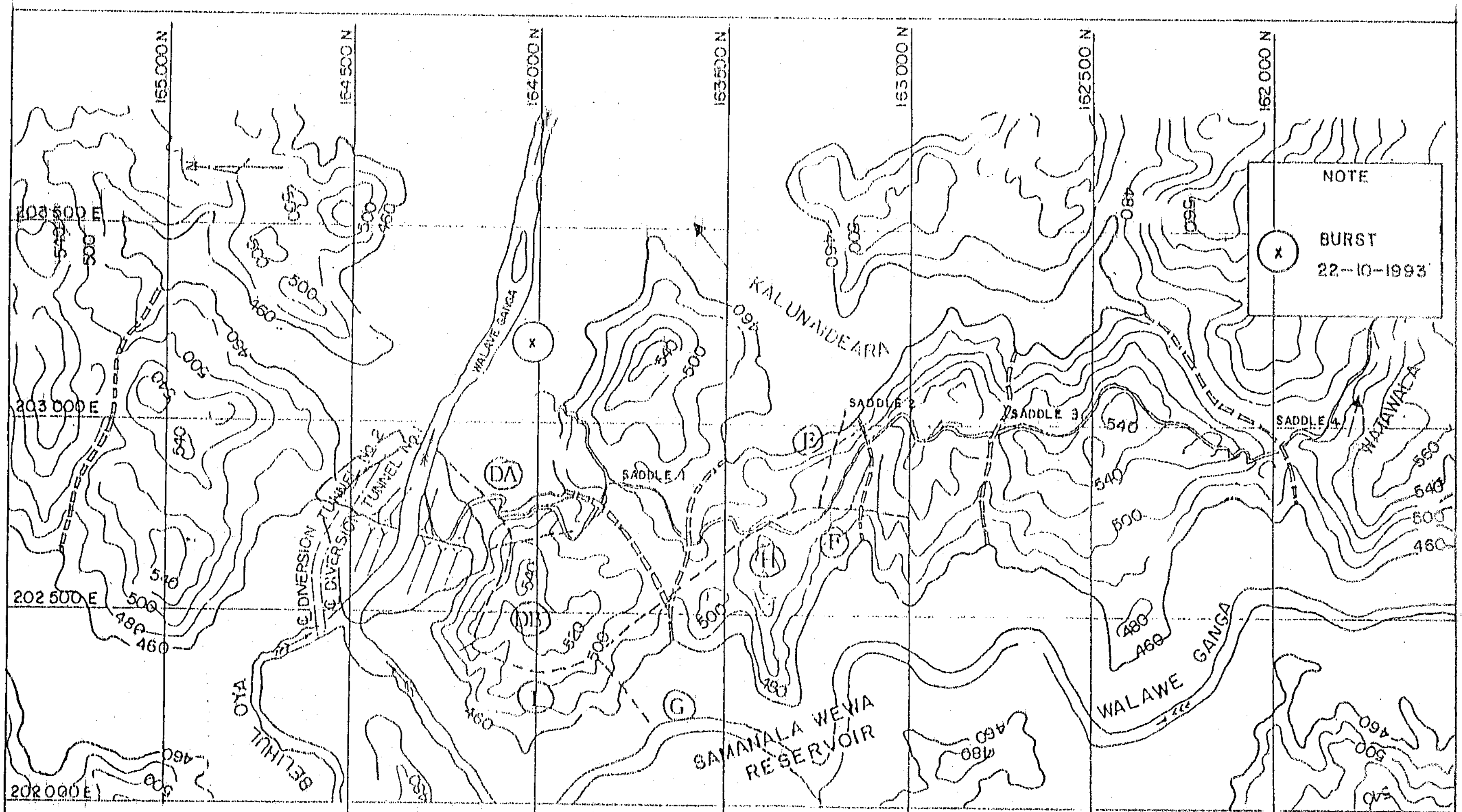


FIG. I



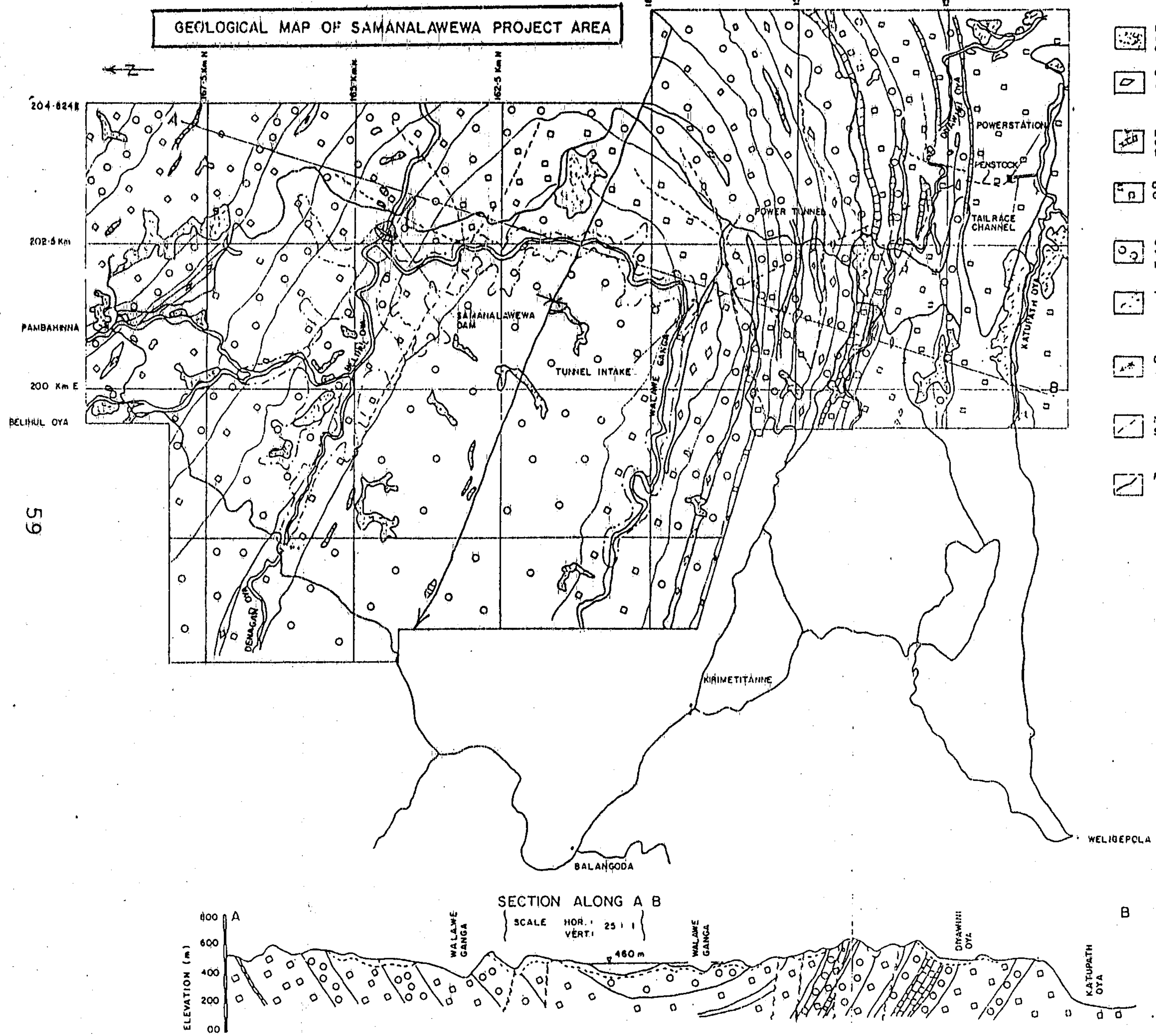
DA, E, G
access adits
F, H, I
Right Bank cutoff
grouting adit
DB, AB
Right Bank cutoff Adits

DESCRIPTION OF SADDLES			
SADDLE No.	DISTANCE FROM DAM (m)	WIDTH OF SADDLE (m)	ELEVATION (m)
1	610	175	480 +
2	1220	310	520 +
3	1550	510	520 +
4	2286	650	540 +
L.B.	850	730	520 +

SAMANALA WEWA
LOCATION OF SADDLES

FIG. 2

GEOLOGICAL MAP OF SAMANALAWEWA PROJECT AREA



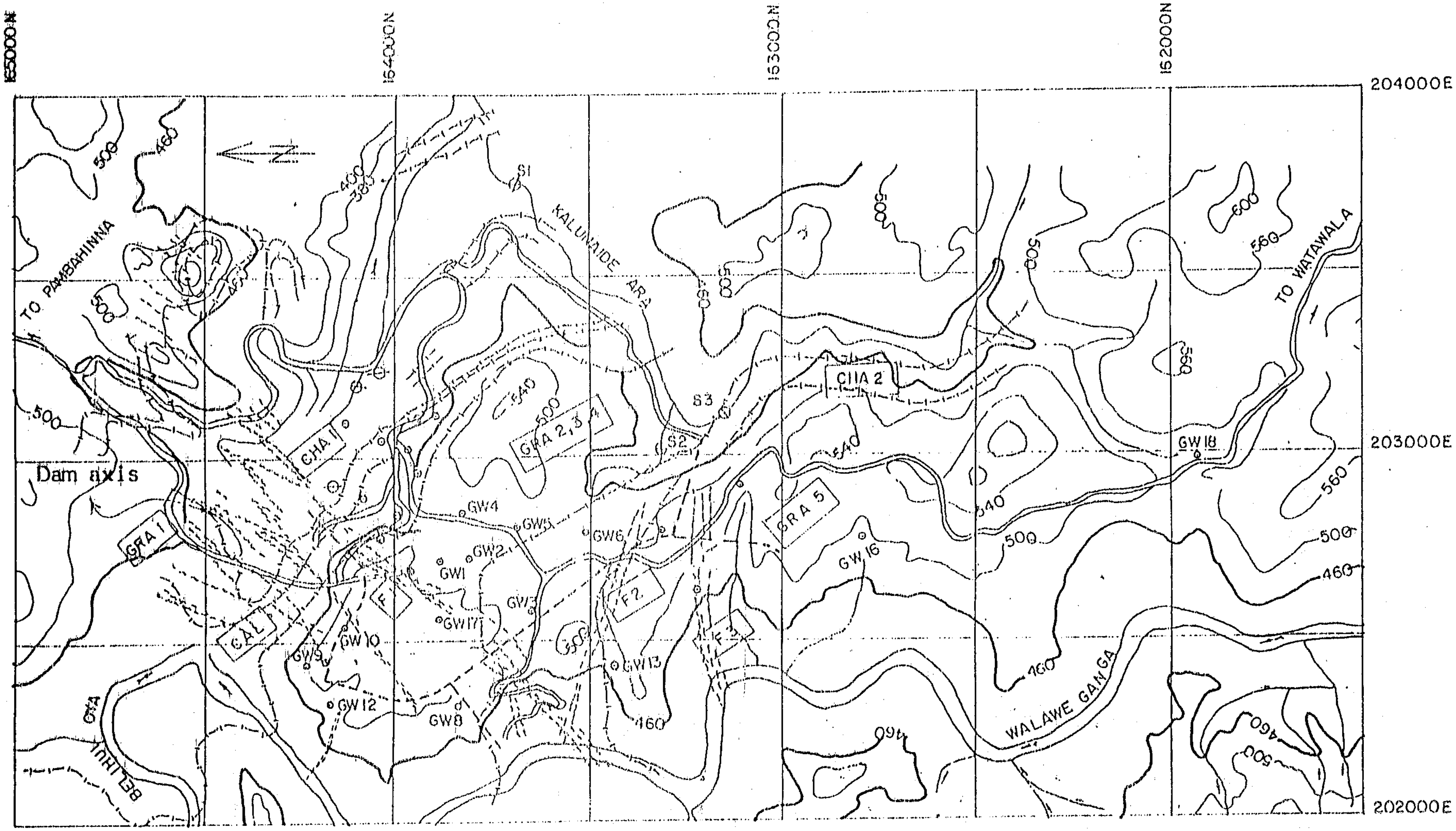
LEGEND

- RECENT SYSTEM ALLUVIAL AND TALUS DEPOSITS. CLAY, LOAM AND SANDY LOAMS INCLUDING GRUSS AND ROCK DEBRIS UP TO 20%
- QUARTZITES INTERBEDED WITH BIOTITE AND GARNETIFEROUS GNEISS.
- MARBLIZED LIMESTONE, DOLOMITE AND CALC GNEISS + IMPURE CRIYSTALLINE LIMESTONE LOCALLY KARSTIFICATED
- CHARNOCKITIC GNEISS, CHARNOCKITIC BIOTITE GNEISS, MIGMATIZED (LOCALLY SILICIFIED)
- GARNETIFEROUS BIOTITE GNEISS, MIGMATIZED WITH SUB LAYERS OF GRANULITES AND KHONDALITES
- TECTONIC FAULTS
- DIRECTION OF PLUNGE AND AXIS OF SYNCLINE
- NORMAL HIGH WATER LEVEL SAMANALAWEWA RESERVOIR
- ROADS

NOTE
EXTRACTED FROM SAMANALAWEWA PROJECT DETAILED PROJECT REPORT TECHNOPROMEXPORT 1978

FIG 3

GEOLOGY OF THE SAMANALAWEWA DAM AND SADDLE AREA

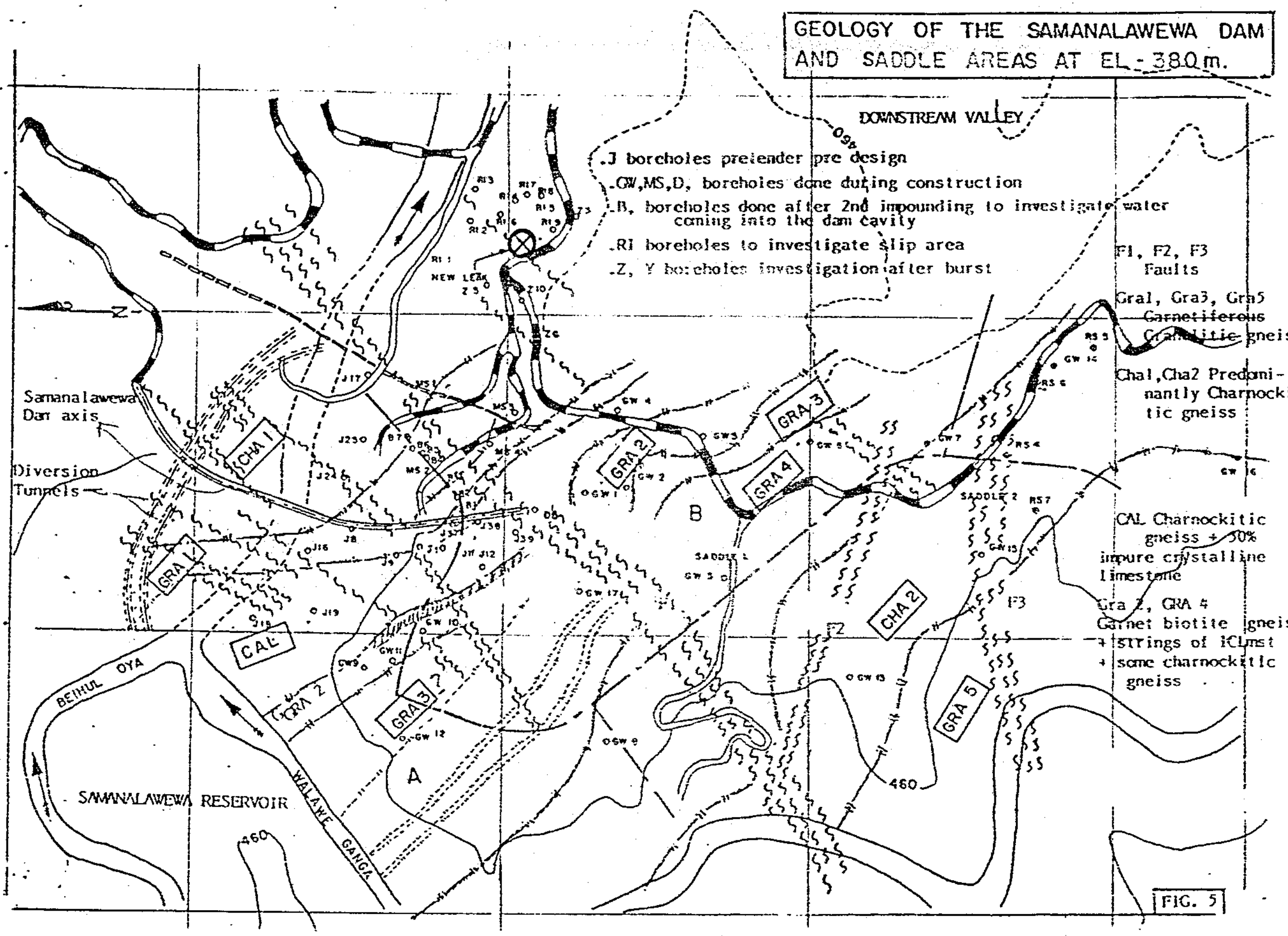


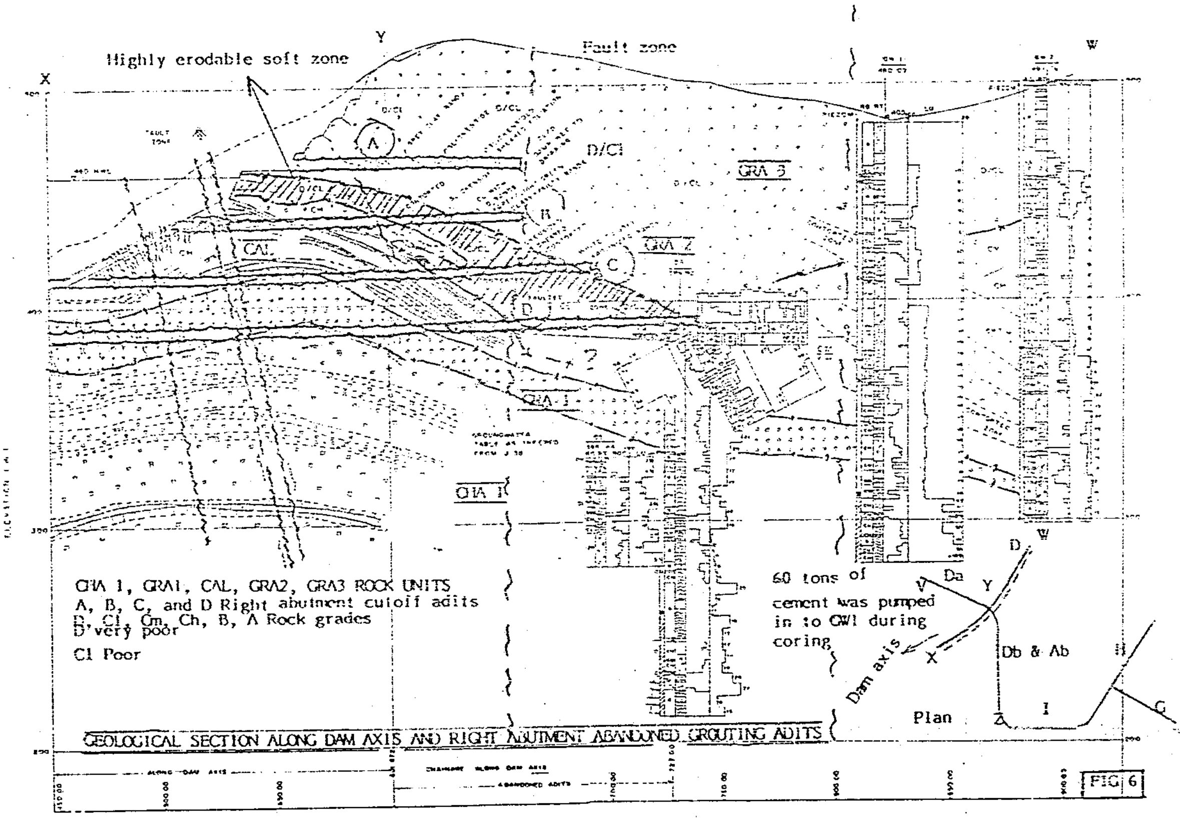
Legend CHA1, GRA1, CAL , GRA2, GRA3, GRA4, CHA2 & GRA5 - Rock Units

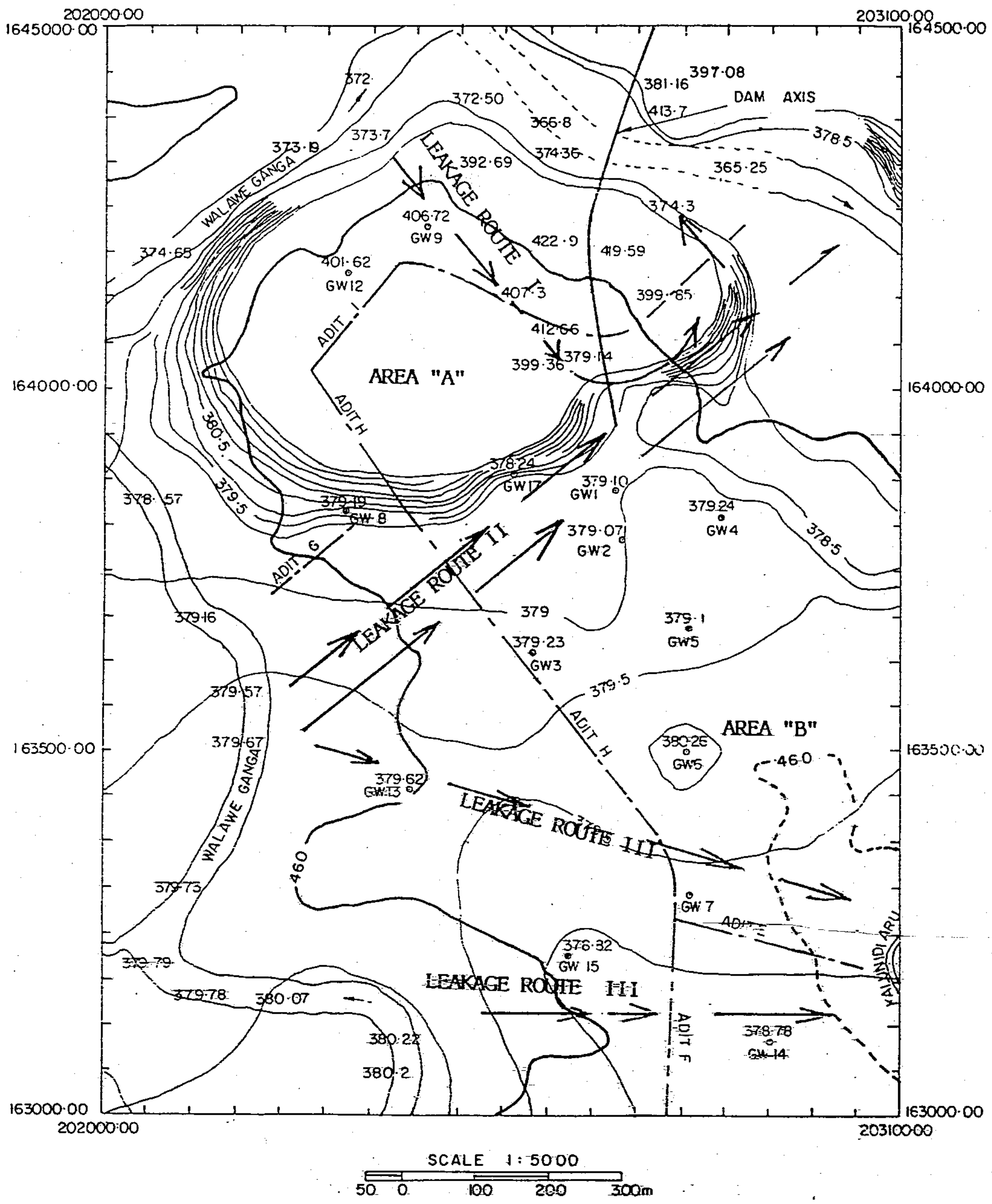
F1, F2 & F3 - Faults

D, MS, GW, Borehole series done during construction

Fig. 4


GEOLOGY OF THE SAMANALAWEWA DAM AND SADDLE AREAS AT EL - 380 m.
DOWNSTREAM VALLEY
.J boreholes pretender pre design
.GW,MS,D, boreholes done during construction
.B, boreholes done after 2nd impounding to investigate water coming into the dam cavity
.RI boreholes to investigate slip area
.Z, Y boreholes investigation after burst
F1, F2, F3 Faults
Gra1, Gra3, Gra5 Garnetiferous Granulitic gneiss
Cha1,Cha2 Predominantly Charnockitic gneiss
CAL Charnockitic gneiss + 50% impure crystalline limestone
Gra 2, GRA 4 Garnet biotite gneiss + strings of (CLmst + some charnockitic gneiss
NEW LEAK
Samanalawewa Dam axis
Diversion Tunnels
CHA 1
GRA 1
GRA 2
GRA 3
GRA 4
GRA 5
CHA 2
CAL
SADDLE 1
SADDLE 2
BEIHUL OYA
WALAWE GANGA
SAMANALAWEWA RESERVOIR
A
B
F2
F3
460
FIG. 5

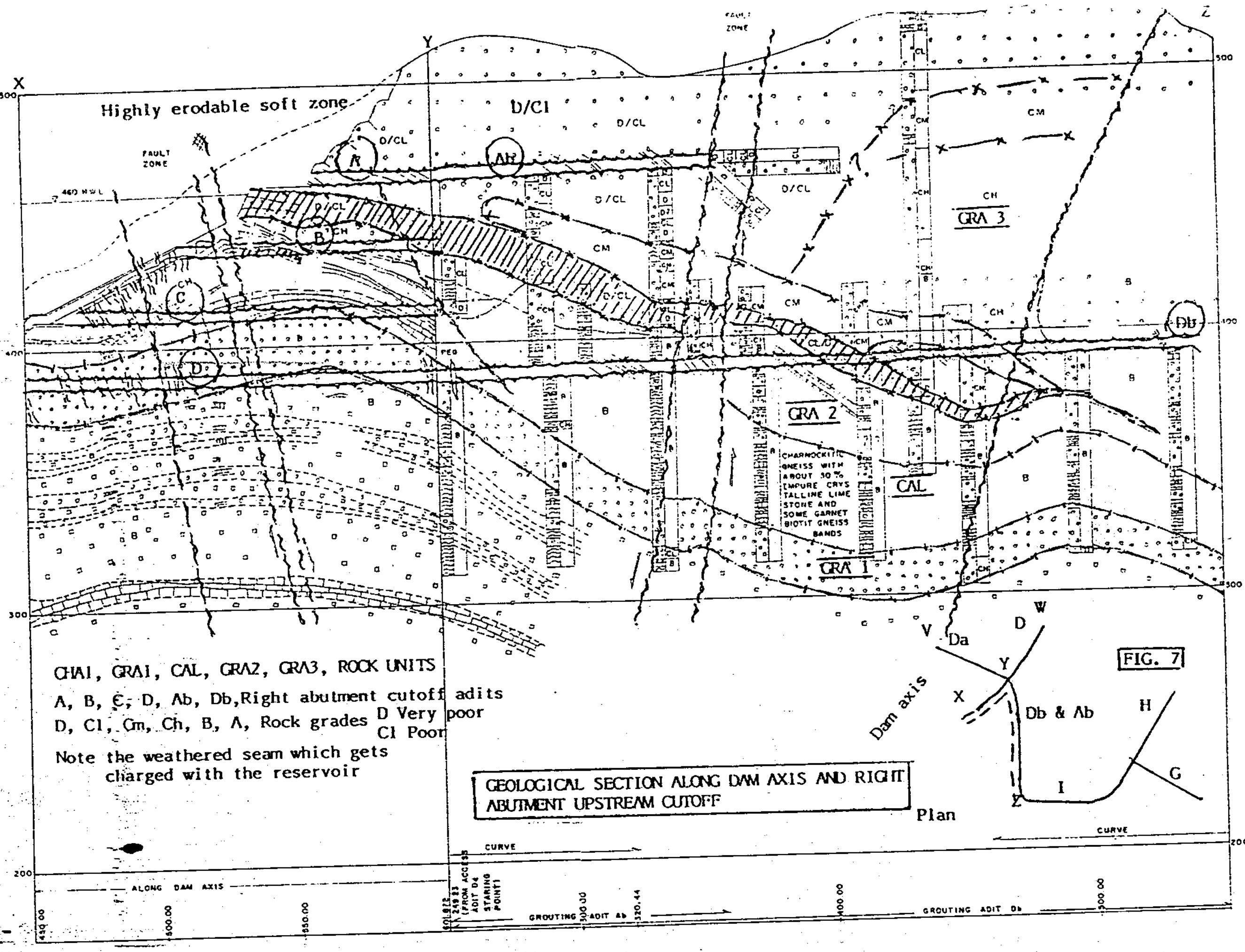

Highly erodable soft zone
Fault zone
X
Y
W
D/Cl
GRA 3
GRA 2
GRA 1
CAL
CHA 1
A
B
C
D
CHA 1, GRA1, CAL, GRA2, GRA3 ROCK UNITS
A, B, C, and D Right abutment cutoff adits
B, Cl, Cm, Ch, B, A Rock grades
D very poor
Cl Poor
60 tons of cement was pumped in to GW1 during coring
Dam axis
Da
Db & Ab
Plan
GEOLOGICAL SECTION ALONG DAM AXIS AND RIGHT ABUTMENT ABANDONED GROUTING ADITS
FIG 6



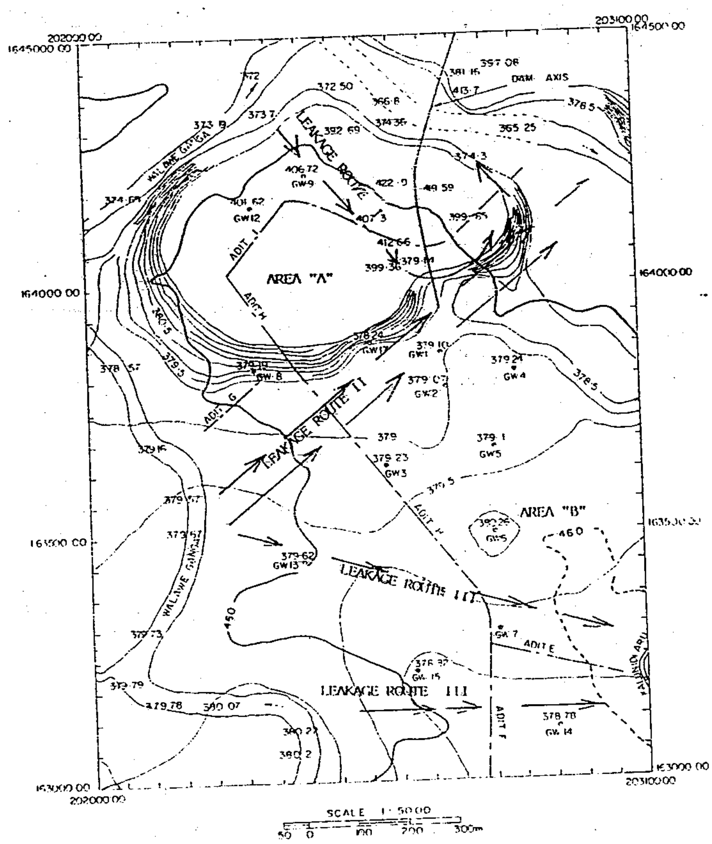
GROUNDWATER CONTOURS OF THE DAM RIGHT ABUTMENT AND SADDLES

Date 29.5.90

FIG. 8

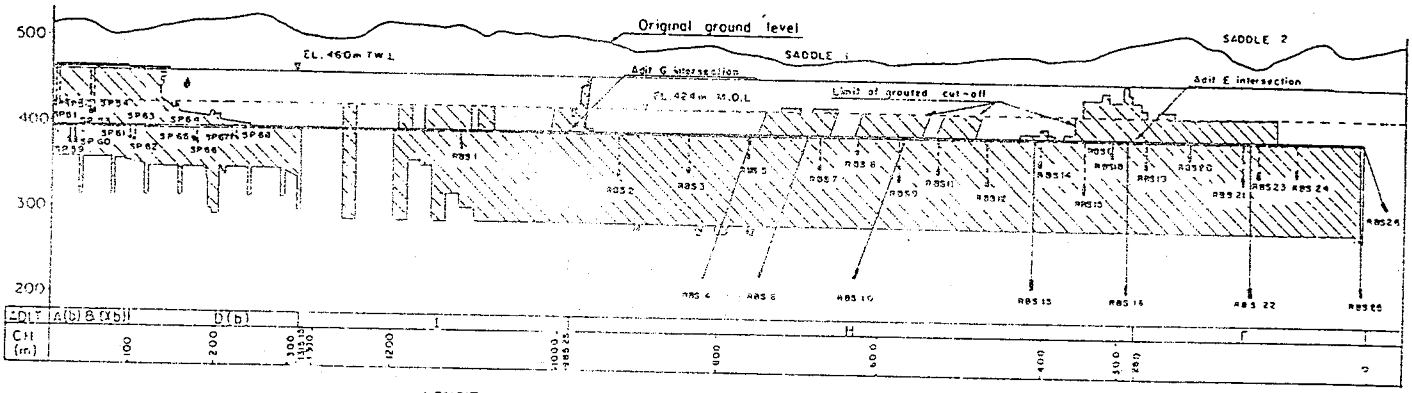


GEOLOGICAL SECTION ALONG DAM AXIS AND RIGHT ABUTMENT UPSTREAM CUTOFF

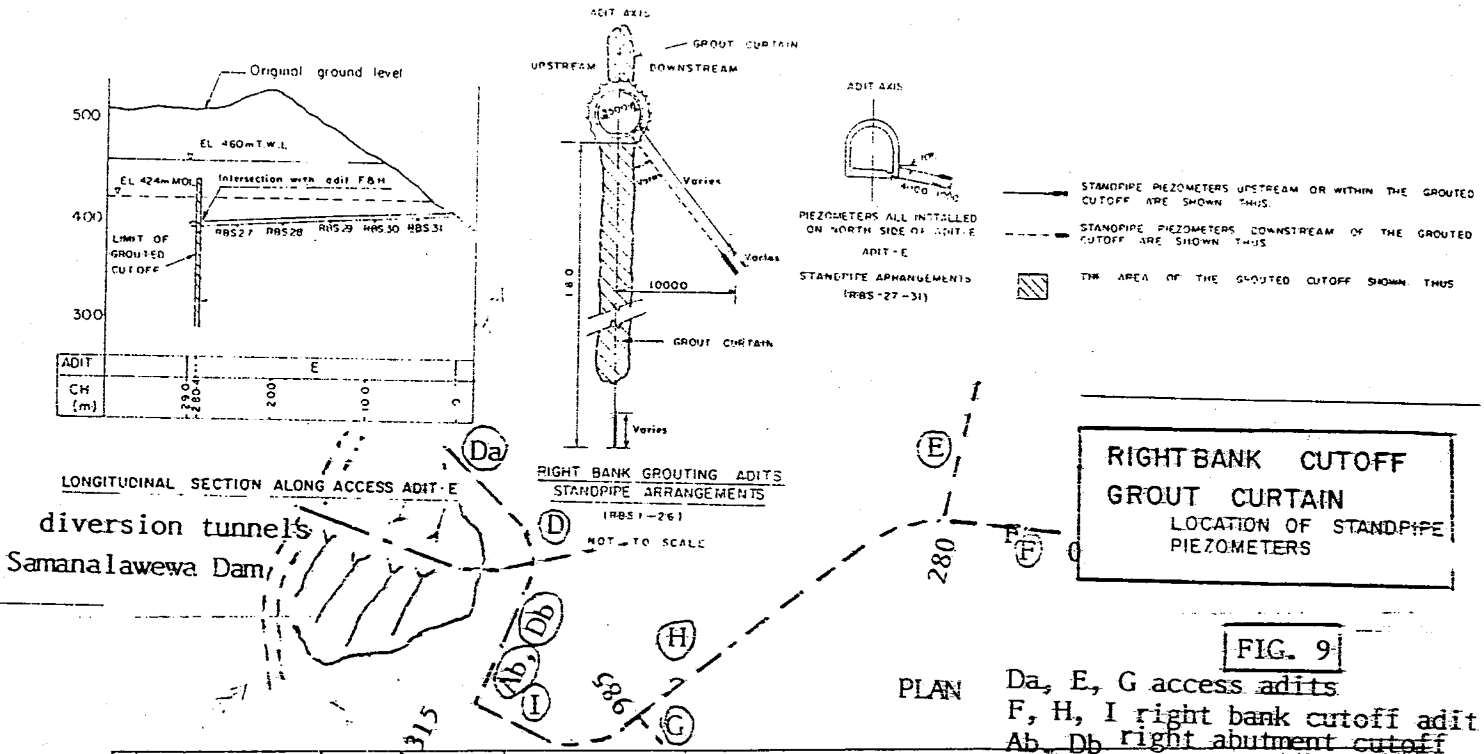


GROUNDWATER CONTOURS OF THE DAM-RIGHT ABUTMENT AND SADDLES

Date 29.5.90



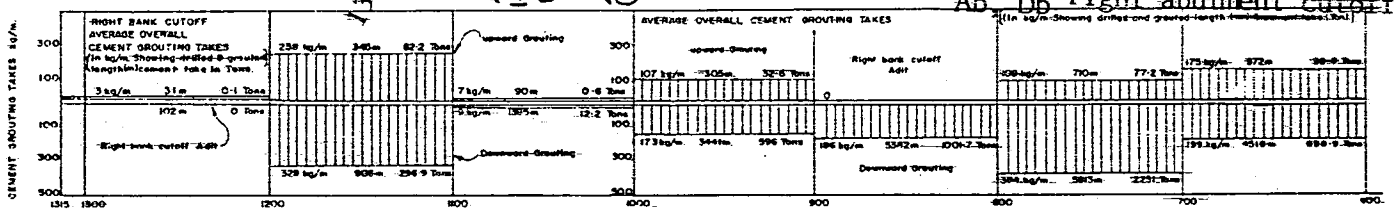
LONGITUDINAL SECTION ALONG RIGHT BANK GROUTING ADITS



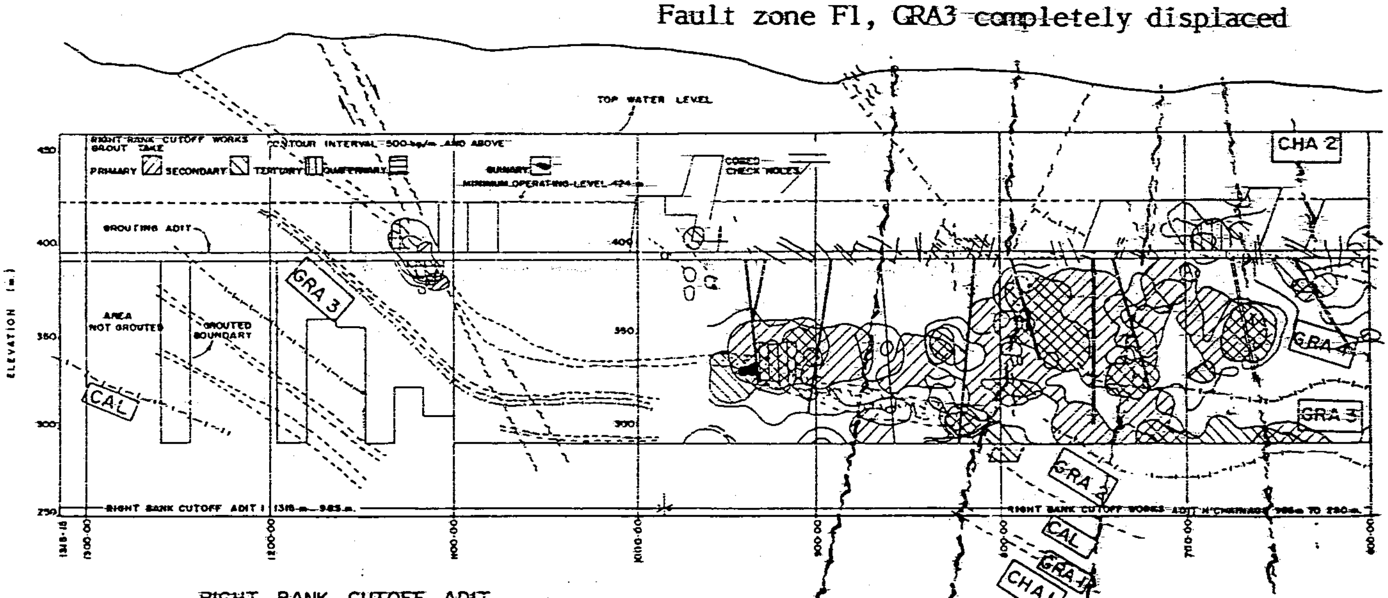
RIGHT BANK CUTOFF GROUT CURTAIN

LOCATION OF STANDPIPE PIEZOMETERS

FIG. 9



Fault zone F1, GRA3 completely displaced

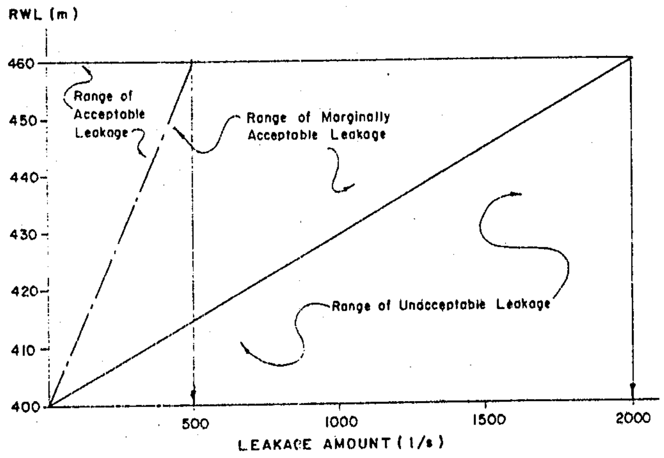


RIGHT BANK CUTOFF ADIT.

Chainage 1315 to 600

GEOLOGY AND CONFIGURATION OF GROUT CURTAIN AND GROUT TAKE

BROAD GUIDELINE FOR LEAKAGE MONITORING

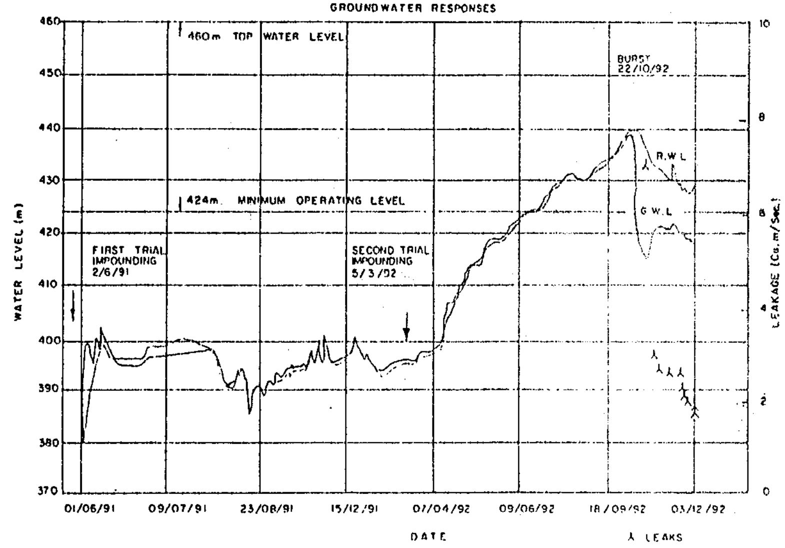


RESERVOIR MANAGEMENT: WATER RELEASES REQUIRED DOWNSTREAM OF SAMANALAWEWA DAM

MONTH	DISCHARGE (m^3/sec)	MONTH	DISCHARGE (m^3/sec)
JAN	1·9	JUL	2·5
FEB	2·3	AUG	2·2
MAR	1·0	SEP	0·0
APR	1·4	OCT	1·7
MAY	2·0	NOV	0·9
JUN	1·3	DEC	2·7

FIG. 10

SAMANALAWEWA HYDRO-ELECTRIC PROJECT



RESERVOIR IMPOUNDING LEAKAGES AND GROUNDWATER LEVEL

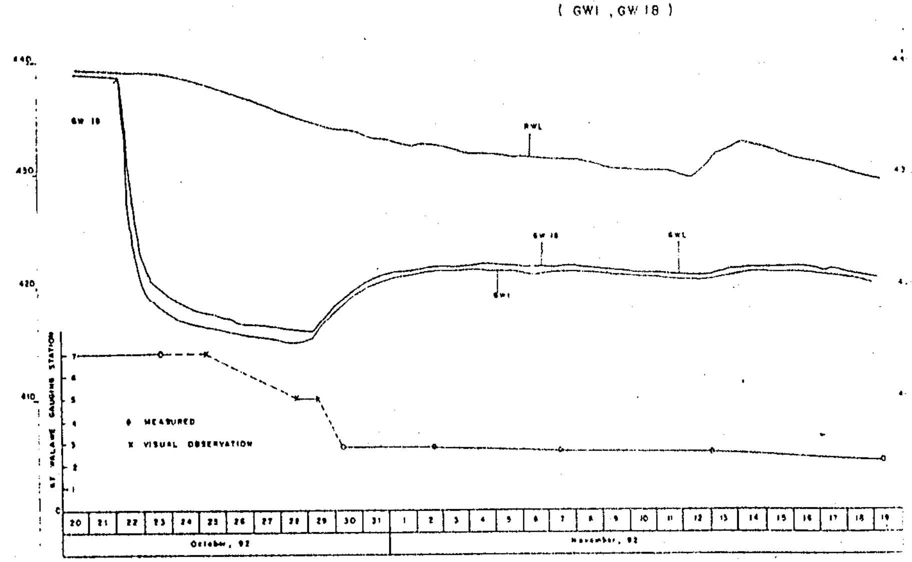


FIG. 11

RESERVOIR IMPOUNDING, GROUNDWATER LEVEL VS RESERVOIR WATER LEVEL

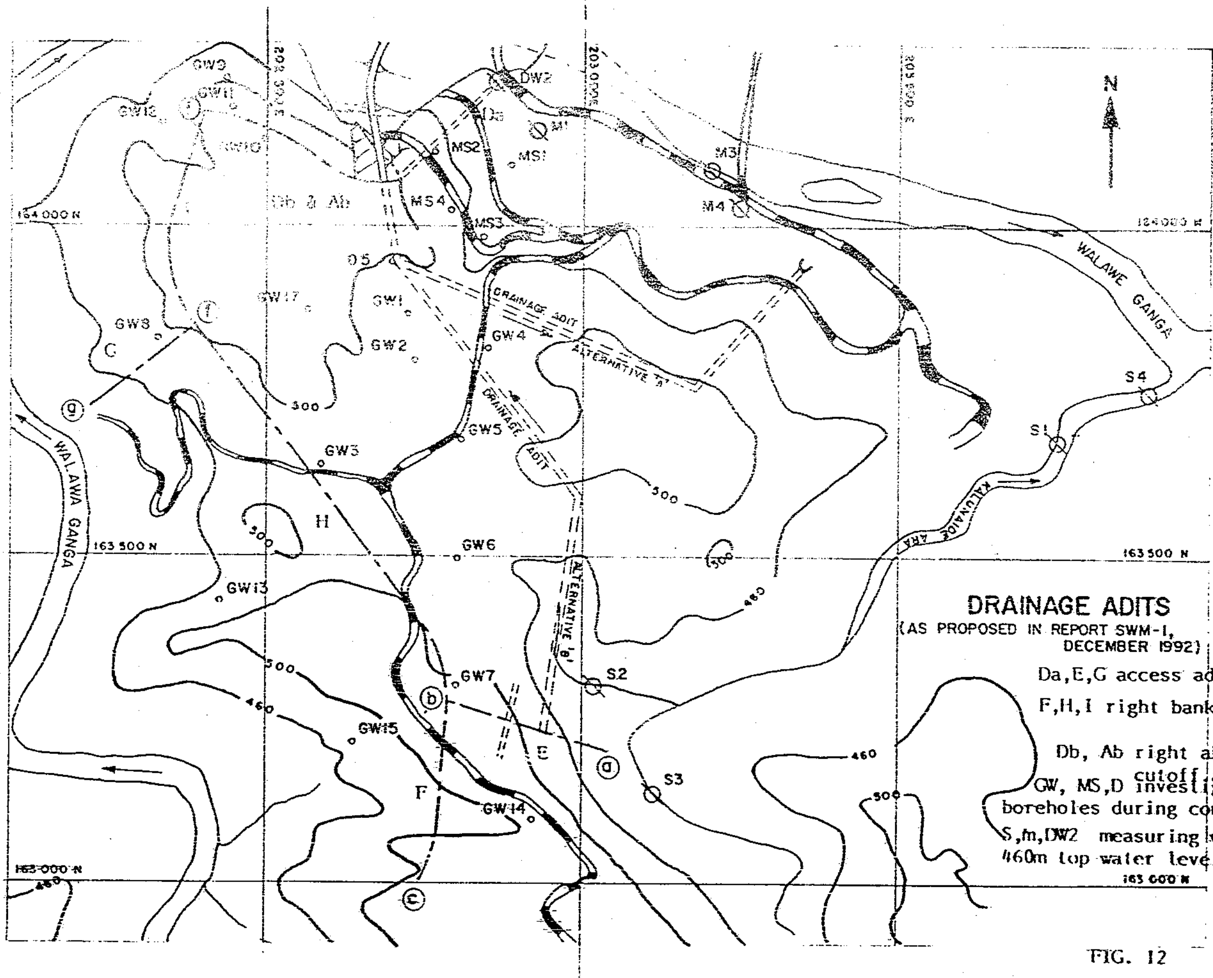
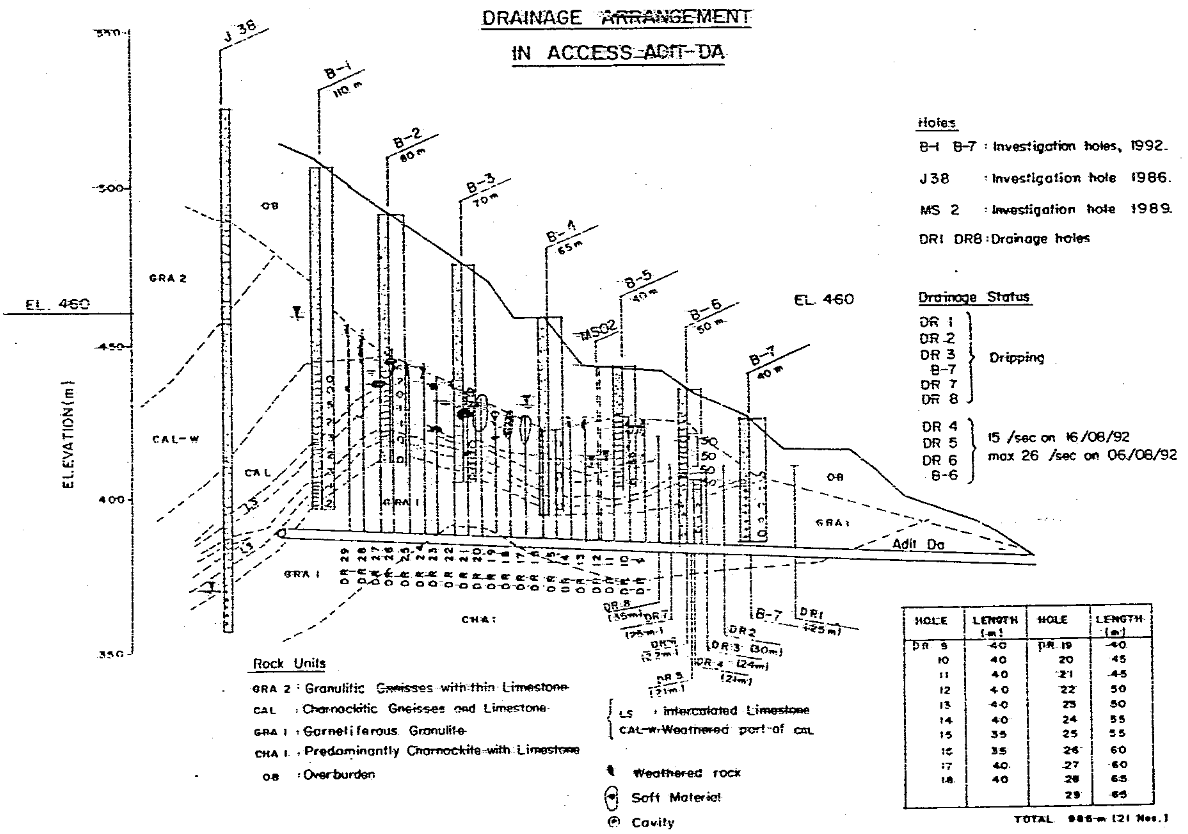


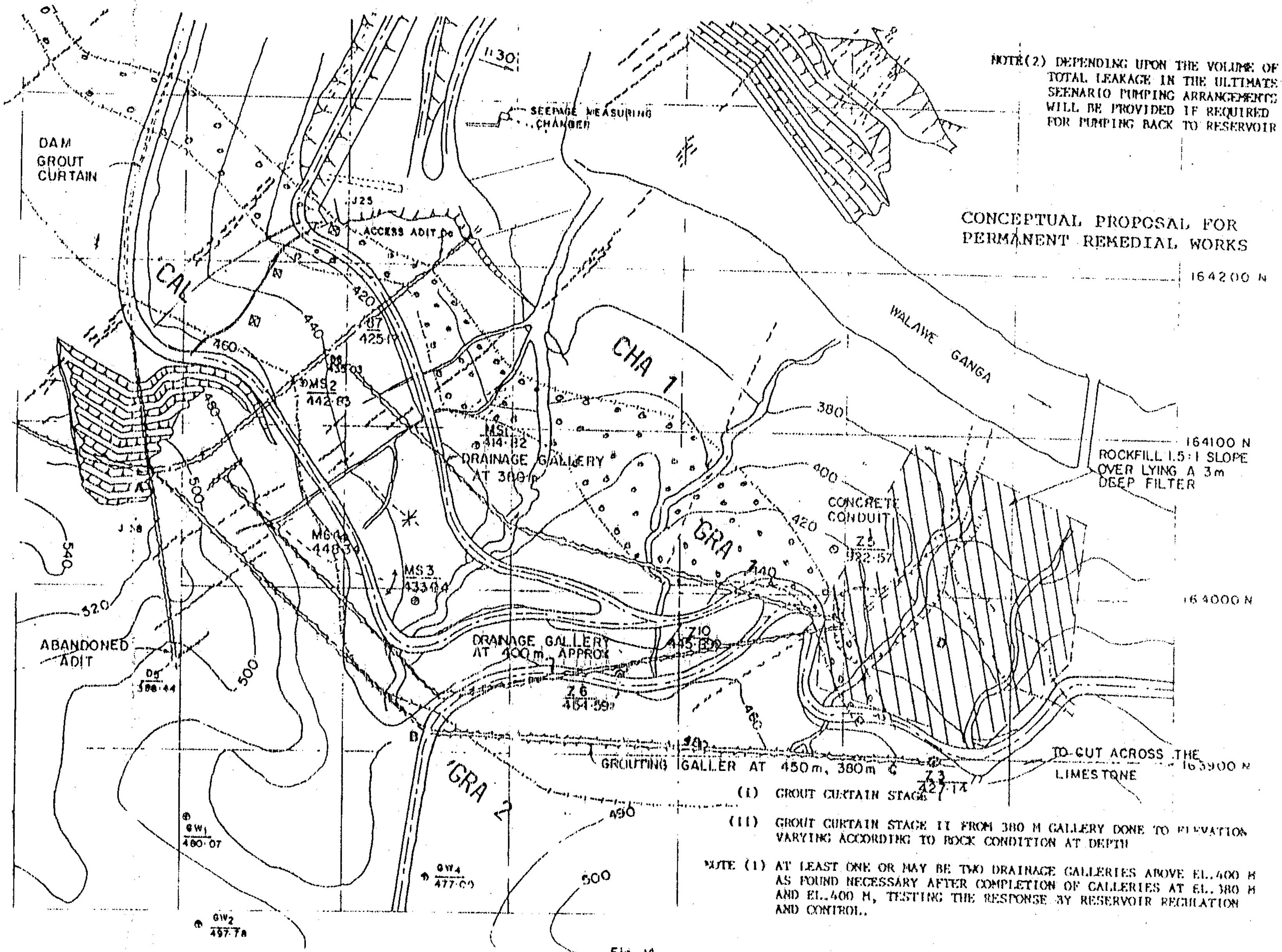
FIG. 12



HOLE	LENGTH (m)	HOLE	LENGTH (m)
DR 9	40	DR 19	40
10	40	20	45
11	40	21	45
12	40	22	50
13	40	23	50
14	40	24	55
15	35	25	55
16	35	26	60
17	40	27	60
18	40	28	65
		29	65

TOTAL 986-m (21 Nos.)

FIG. 13



CONCEPTUAL PROPOSAL FOR PERMANENT REMEDIAL WORKS

FIG. 14

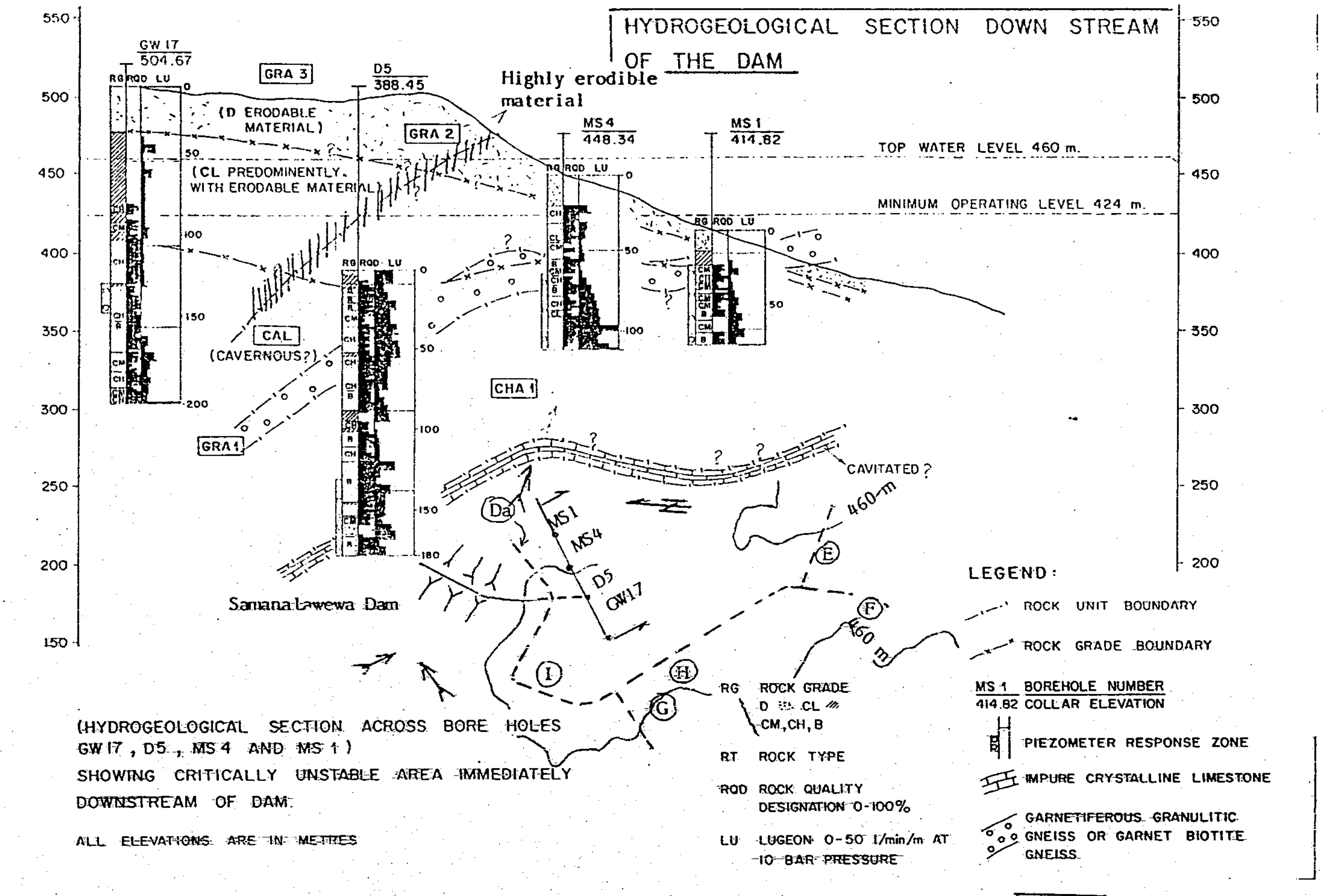
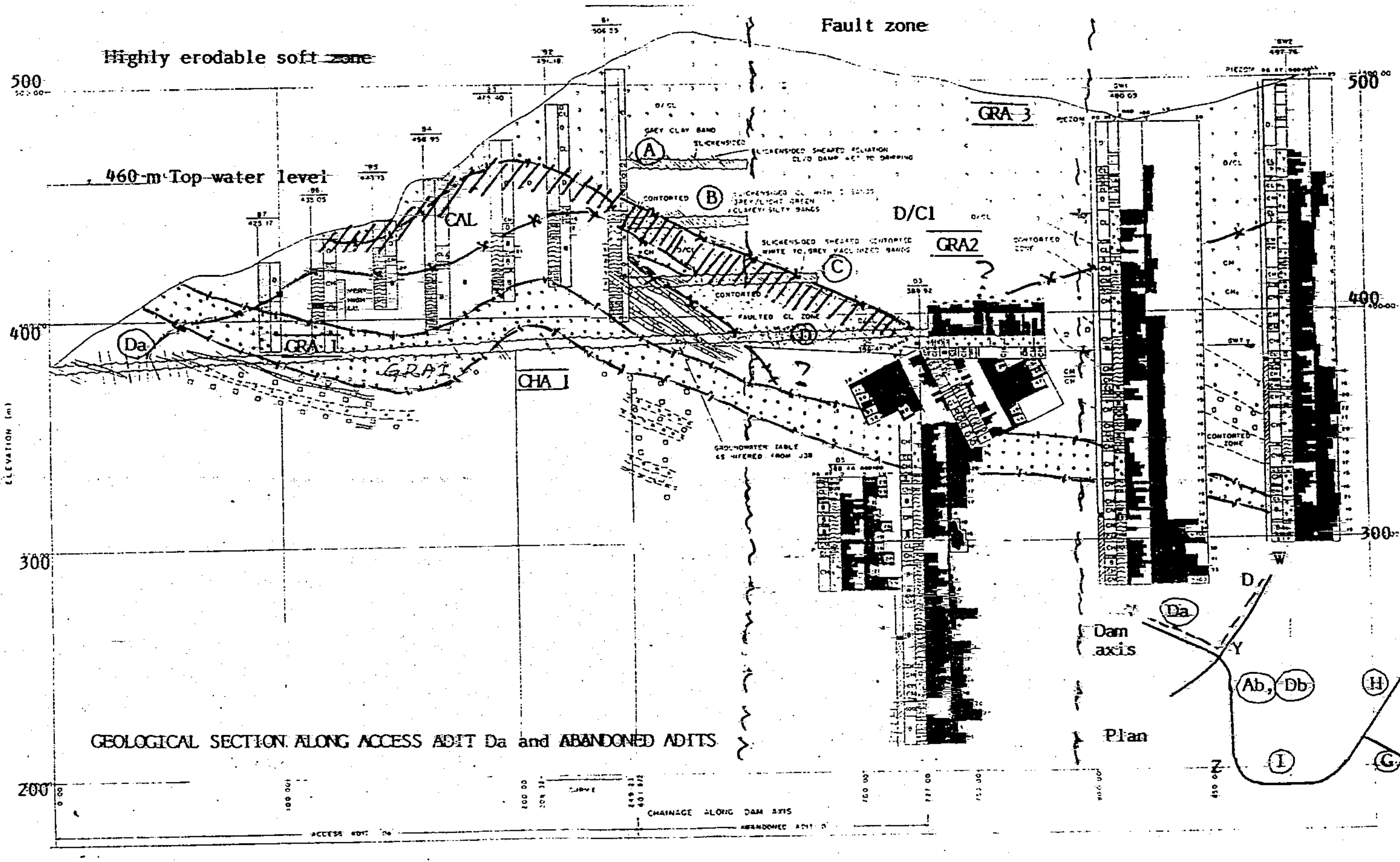

HYDROGEOLOGICAL SECTION DOWN STREAM OF THE DAM
Highly erodible material
GW 17 504.67
D5 388.45
MS4 448.34
MS 1 414.82
RG RQD LU
GRA 3
GRA 2
(D ERODABLE MATERIAL)
(CL PREDOMINENTLY WITH ERODABLE MATERIAL)
TOP WATER LEVEL 460 m.
MINIMUM OPERATING LEVEL 424 m.
CAL
(CAVERNOUS?)
CHA 1
GRA 1
CAVITATED ?
Samanalawewa Dam
460 m
LEGEND:
ROCK UNIT BOUNDARY
ROCK GRADE BOUNDARY
MS 1 BOREHOLE NUMBER
414.82 COLLAR ELEVATION
PIEZOMETER RESPONSE ZONE
IMPURE CRYSTALLINE LIMESTONE
GARNETIFEROUS GRANULITIC GNEISS OR GARNET BIOTITE GNEISS
RG ROCK GRADE D CL CM, CH, B
RT ROCK TYPE
RQD ROCK QUALITY DESIGNATION 0-100%
LU LUGEON 0-50 l/min/m AT 10 BAR PRESSURE
(HYDROGEOLOGICAL SECTION ACROSS BORE HOLES GW 17, D5, MS 4 AND MS 1)
SHOWING CRITICALLY UNSTABLE AREA IMMEDIATELY DOWNSTREAM OF DAM
ALL ELEVATIONS ARE IN METRES

FIG. 15


Highly erodable soft zone
460 m Top water level
Fault zone
GRA 3
GRA 2
D/C1
CAL
GRA 1
CHA 1
Dam axis
Plan
ELEVATION (m)
GEOLOGICAL SECTION ALONG ACCESS ADIT Da and ABANDONED ADITS

PROBLEMS ENCOUNTERED IN TUNNEL CONSTRUCTION (CASE HISTORIES)

A. K. D. N. Atukorale

INTRODUCTION

In most of the hydropower projects in Sri Lanka tunnels have been used as part of the water conveyance system. In selecting a tunnel route the subsurface investigations are done to enroute the proposed tunnel trace in sound rock because tunnelling through weak rock is generally difficult and time consuming. But in some instances these weak zones cannot be avoided completely and in such situations it is necessary to minimize the extent of such features. The other important factor in selecting a tunnel trace is the stability of tunnel portals. It is generally prudent that the bulk excavation for tunnel portals should be minimized and the excavated slopes should be stabilized. In some instances it is necessary to construct a reinforced concrete portal structure to prevent possible slope failures.

CANYON POWER TUNNEL (From January 1979 to September 1981)

The Canyon power tunnel 4.3 km long and 3.1 m finished diameter having a modified horse shoe shape is situated on the right bank of the Maskeliya Oya. The tunnel intake is located at the downstream of Maussakelle dam and discharges water to the power station situated near Canyon dam. The layout of the features are shown in Fig. 1. The contract for the construction of the civil works of the entire project was awarded to the State Development & Construction Corporation (SD&CC) of Sri Lanka.

The construction of the power tunnel was inaugurated on February 20, 1978 with the excavation of the 400 m long access adit. The excavation of the main tunnel commenced in January 1979.

The bulk excavation for the outlet portal also commenced during the same period. The construction drawings issued showed 11,000 m^3 of earth excavation and 6,000 m^3 of rock excavation. As excavation proceeded it was revealed that the rock encountered at predicted elevation was of highly joined nature and not suitable for tunnelling through it. Subsequently it was confirmed that sound rock was another 10m below the anticipated level. As a result the extent of bulk excavation was lowered further by 10m to avoid the 30m of tunnelling through soft rock. The additional bulk excavation anticipated due to this revision was 7000 m^3 of earth and 7600 m^3 of rock excavation.

The excavation continued according to the revised drawing when a large cleavage almost encircling the boundary of the portal excavation was observed after the cyclone rains on November 25, 1978. The width of the cleavage was 0.6m at certain places and the maximum depression was about 1.0m. The visible depth of this crack was nearly 10m at the apex. The sliding mass was approximately 60mx70m which had an estimated quantity of 30,000m^3 of moving mass.
As a result of this the location of the outfall had to be shifted to a new site and it was relocated adjacent but out side the moving mass where the rock was exposed. The excavation for the tunnel commenced at the new location after stabilizing the rock face with rock bolts and constructing a reinforced concrete portal structure. Full face excavation was done with 0.6m pulls, 22 nos of steel ribs were installed at 0.9m spacing before the tunnel entered the hard ground where the excavation continued with 2.4m rounds of full face blasting.

The first 17m of the intake drive had to proceed with heading and benching method using steel ribs at 0.5m spacing due to the predominant seepage through the rock and the limitations imposed to control the bulk excavation as the site was located in the right abutment of the Mausakelle dam. At the adit upstream drive a weak zone was encountered which extended to 37m where full face blasting with 0.6m to 0.9m rounds were done and steel ribs was installed at 1.0m spacing.

Apart from the above obstructions, the Contractor had to face two major bad tunnelling stretches, one in the adit downstream drive and the other in the outfall drive. The total length of the tunnelling in weak ground was only 4.85% of the total tunnel length. The rest 95.15% of the tunnel was in good rock and proceeded with full face blasting generally with 2.4m rounds. During excavation of the tunnel, decision was made to divert the tunnel route twice to avoid further delays in tunnelling through weak zones and to avoid crossing of possible weak zones at length.

After completing 310 m of tunnelling in the adit downstream drive the signs of steadily increasing seepage water through rock joints and the deterioration of the quality of rock was evident. As a result the depth of rounds was reduced to 1.2m. At Stn.20+74 while drilling for the blast muddy water came through most of the drill holes and after the blast it was noticed that the tunnel face consisted of a matrix of boulders and highly weathered material with substantial seepage. The Contractor could not take the precautionary measures in time and the tunnel crown collapsed progressively.

As a result of this the Contractor had to make a considerable effort to stabilize the collapse. Finally 50mm x 50mm angle irons were driven ahead of the tunnel roof as forepoling before the removal of collapsed material was started. After removing of collapsed material the drive advanced with heading and benching method and steel ribs were installed at 0.6m spacing. Reinforced concrete laggings were used and the space behind the laggings was filled with rubble.

In this bad stretch the Contractor could achieve only an average progress of 2m per week for a period of five months which is very unsatisfactory. This particular zone was not so highlighted in the technical report. Since no signs of improvement was evident even after 42m of tunnelling, a decision was made to suspend the tunnel drive and to investigate the extent of this weak zone. A horizontal cored drill hole made from the tunnel face indicated that another 73m of similar ground is present before entering the sound rock. With this information it was decided to carry out further investigations in order to find out possibilities of minimizing the tunnelling through such bad ground. Three more holes were drilled on the right hand side tunnel wall fanning out at different angles. The total core drilling involved in this investigation was 602m including a vertical hole at the tunnel face. The coclusion of these investigations was to divert the tunnel route on which only 27m of bad tunnelling is required. As a result of this decision a 42m of tunnel length driven through bad ground with greatest difficulties had to be abandoned.

While excavation was in progress on the realigned tunnel trace the predicted weak zone consisted of a completely weathered homogeneous kaolinized clay and 27m long was encountered. In this stretch the excavation was done in heading and benching method and steel ribs were placed at 0.6 m spacing. As the bearing capacity of this material seem to be very low invert girders were introduced to all steel ribs and the tunnel invert was concreted with 25 mm diameter steel reinforcement at 300 mm spacing in both directions. In addition to this the side walls were also concreted upto the spring level and reinforced concrete laggings were used above the spring line.

At the downstream drive a fault was encountered at Stn. 38+70 which consisted of moderately to highly weathered rock. This zone was initially excavated with full face blasting with 1.2m to 0.8m rounds subjected on the weathering state of the rock encountered. Steel ribs were placed in this stretch totalling to 40 Nos. at 0.92 m spacing. At Stn.38+34 the tunnel face was flooded with muddy water issuing from left corner of the invert. The legs of the steel ribs of this stretch were disturbed and subsequently concrete placed surround the steel ribs.

Once again the driving of the tunnel face was suspended on May 31, 1980 and investigations were commenced to identify the nature and the extent of the bad stretch. Three horizontal holes were drilled to a total length of 86 m of which one was parallel to the tunnel centre line and the other two fanning out on either side. Investigations revealed that this stretch continued for another 15 m. Water started gushing out through these drill holes carrying considerable amount of silt. The rate of water flow through these holes was 11 litres per second at the beginning and flow reduced to a uniform rate of 4 litres per second within a short period. The silt ejected by this hole was recorded to be about 34 cu.m. during the period from June 6, 1980 to October 25, 1980.
While the excavation continued by heading and benching method with steel ribs at 0.6 m. spacing the tunnel roof collapsed at Stn.37+81 bringing approximately 200 m^3 of mud and boulders into the tunnel. When this failure occurred the tunnel face had advanced to Stn.37+73. A cavity was found located on the right hand side of the tunnel roof having approximate dimensions of 9 m long 7.5m wide and 6.0m in height. A dried stream running parallel to the tunnel was observed within the cavity about 3m to the right at soffit level. Following this roof collapse the water flow from the cored hole at Stn.38+29 stopped immediately. The rate of water drained from this cavity was at 6 litres per second and carried muddy water with silt.

Core drilling was again instructed after suspending the tunnel drive on August 26 1980. These investigations revealed that this condition continued for a longer length as the tunnel was running parallel to the strike. After the investigations were completed the tunnel was rerouted within the sound rock. The excavation of the tunnel was completed on September 11 1981.

2.VICTORIA POWER TUNNEL (From Sept 1980 to Nov 1982)

The Victoria power tunnel 5.7 km long and 6.2 m finished diameter is situated on the right bank of the Mahaweli ganga. The tunnel intake is located immediately upstream of the Victoria dam and the downstream portal is located above the power station at the start of the surface penstock. The layout of the features are shown in Fig. 2. The contract for construction of the Victoria power tunnel was awarded to the Joint Venture of Balfour Beatty Construction Ltd. and Edmund Nutall Ltd. (BBN) of Britain.

The main site investigation for the tunnel contract was carried out during the period from November 1978 to February 1979 which consisted of one bore hole at Power intake and seven along the tunnel trace of which three were sunk to locate a suitable location for the surge chamber. Generally the rock along the tunnel length is very sound with low permeability. It was also noted that the tunnel will cross a layer of crystalline limestone twice. Serious doubts about the competence of this layer was doubted due to the results of two bore holes BT 201 and BT 204 of which BT 204 encountered artesian water at a position 8 m below the tunnel line. There was much concern that considerable quantities of water might be encountered in the limestone layer.

At the downstream drive significant water was encountered from the probe holes on three occasions during the first four months of excavation. On two of these occasions the water flow was controlled by grouting through the probe holes and additional holes.

The crystalline lime stone was encountered at CH5570 and throughout the extent it was sound and almost dry contrary to the prediction. The drive had progressed 600 m when indication of water ahead was evident from the steadily increasing degree of wetness as the drive advanced. A probe hole drilled at CH.5321 to 29 m ahead encountered water at 18 m. The water flowed barrel full from the hole and the pressure indicated was of the order of 10 bars. Fissure grouting was done to stabilize the zone and excavation resumed after about 233 t of grout had been injected . A major rock fall occurred on July 31, 1981 after a couple of rounds have been pulled. The rock fall was accompanied by an inrush of water estimated at over 500 liters per second. The heading was completely choked by the rock fall which was a mixture of large boulders and highly weathered material. At this stage it was not possible to identify the extent of the collapse. In the course of 48 hours after the rock fall the water flow was reduced to about 100 liters per second. In the mean time it was decided to investigate the ground ahead by drilling two horizontal holes fanning out on either side of the heading. These were drilled to a depth of 100 m and revealed a wide fault crossing the tunnel at an angle of about 30^{0}. The removal of rock fall material which estimated to approximately 1000 m^3 was removed within a period of two months and revealed a chimney to the left side of the tunnel crown ascending to an uncertain height probably to the order of 30 m. The neck approximately 4 m X 5 m of the chimney existed immediately above the tunnel crown through which occasional rock falls continued to occur.

For safety reasons it was decided to backfill the neck of the chimney and the heading below with concrete and to tunnel through this plug. A 250 mm diameter hole was drilled through the plug in to the chimney and concrete placed to form the plug . The tunnelling through the plug with a crown pilot heading began two days after placing the plug. The top heading was supported with arch ribs and extended to new ground. This was a conglomeration of highly weathered material with occasional pockets of sound rock with traces of slicken sideds. Traces of cement from the grouting operation was also detected but it was apparent that this had contributed little to improve the strength of material. A diagram of rock fall location is shown in Fig. 2.

While this work was in progress at the tunnel face other options were being investigated. The core drilling that had been carried out to investigate the fault zone had indicated that the tunnel could possibly recross the projected line of the fault beyond the bend near the surge chamber. In view of the delays that had already been accumulated this had serious programme implications. It was therefore decided to investigate the feasibility of diverting the tunnel so that it could be avoided entirely.Investigation drilling carried out from the surface confined more accurately the orientation of the fault line and the soundness of the overline limestone of the proposed relocated tunnel trace.

At the end of the investigations i.e after six months the decision was made to abandon 120 m of existing tunnel including the ground hard won from the fault to realign the tunnel and to relocate of the surge chamber.

This decision meant that the surge chamber was to be sited over on already excavated section of the tunnel. Excavation of the pilot shaft for the surge chamber could now proceed. The original plan had been to drive the outfall heading to a point just beyond the surge chamber where the tunnel excavation could be discontinued and resources moved to the main adit. There the excavation was to continue on both drives using two sets of equipment. The accumulated programme delays now amounting to about 15 months and concern developed about the uncertainty of encountering additional water bearing features. It was decided to continue the driving from the downstream end beyond the surge chamber. To resource this a three boom hydraulic drill rig and two Cat 980 front end loaders were air freighted.

In order to allow simultaneous mucking of the surge chamber and the downstream drive the tunnel was enlarged at the base of the surge chamber and a temporary split wall was constructed at the base of surge chamber to operate mucking independently.

In the adit north drive a major fault that had been anticipated was encountered. The ground was supported with shotcrete and mesh. another zone of unsound rock further on was supported with steel arch ribs and steel laggings which was a slower exercise than the shotcrete operation.

The tunnel excavation was competed in November 1982 which was approximately ten months behind the original construction programme.

SAMANALAWEWA POWER TUNNEL (April 1987 to March 1990)

The Samanalawewa Power Tunnel 5.4 km. long and 4.5 m finished diameter is situated across the southern-limb of a large symform in the middle way of the Mahawelatenna - Kalthota escarpment. The layout of the features are shown in Fig. 3. The Contract for construction of power tunnel was awarded to the Balfour Beatty Construction International Ltd. (BBCIL) of Britain.

The site investigation for this project had been done by several organizations since 1966 and the content of these reports have been used to assess the anticipated geological conditions for the design of the tunnel route. According to the reports about 70 percent of the tunnelling would have been in good rock with high strength. In the zones of discontinuities and in limestone the groundwater breakthrough into the tunnel was expected upto 100 litres per second which would reach 600 litres per second from the entire tunnel length.

At the upstream drive heavy water inflows were encountered either during drilling of probe holes or immediately after the face was fired. At Stn.48+10 when the face hit water at a rate of 55 litres per second, additional holes were drilled to drained the surrounding area and core drilling was done to investigate the ground ahead of the face. After the investigation was completed the face was advanced by heading and benching method and the ground was stabilized with steel ribs and shotcrete with mesh. As the excavation progresses to Stn.46+99 the heading tunnel encountered water inflow of 230 litres per second. Once again a large inflow was encountered at Stn.45+09 which increased to 300 litres per second. The majority of the tunnel length in this stretch was excavated by heading and benching method and steel ribs with mesh reinforced shotcrete was applied to stabilize the ground.

At Stn.42+69 the tunnel intersected a fault infilled with solution features. Excavation to Stn.42+18 continued in brecciated rock mainly by heading and benching method with steel ribs, spile bars, rock bolts and two or three layers of shotcrete which took 10 1/2 weeks. The investigations carried out suggested that the karsted fault zone is running parallel with the tunnel line. Due to the slow tunnelling progress arisen from these conditions encountered further investigations were carried out in the area of Stn.42+69 to Stn.34+24. The results of these investigations showed that similar conditions would be encountered further more on the original tunnel route. These investigations indicate that any tunnel realignment should be towards the west of the original route. This would bring the route closer to the Russian alignment along which the most sub surface information is available. After comparing the anticipated geological conditions along the original route and on the realigned route a decision was made to divert the tunnel because the realigned route was likely to reduce the stretch of tunnelling in weak rock.

Between Stn.36+97 and Stn.36+67 large cavities were encountered in the tunnel walls and roof and hit water at Stn.36+67 at a rate of 126 litres per second. The cavities encountered were filled with dry shotcrete blown into them.

A series of roof collapses were occurred in both drives specially immediately after the face was fired. To overcome the progressive collapse of the roof and formation of a chimney above the roof, the pile of collapse material was shotcreted with the rest of the face. After the collapse was arrested temporarily 150 mm x 50 mm steel channels were driven along the periphery of the tunnel crown as forepoling. The collapse material was then removed gradually and a suitable support system (usually steel ribs) was installed. In most of these cases the excavation proceeded by heading and benching method.

Several wall collapses behind the face also occurred a few days after the completion of the support system. In these areas the ground was completely weathered and quite dry during excavation. While the excavation progressed the water from the services started percolating into the ground and the invert started deteriorating due to the absorption of water. As a result the failure of the shotcrete shell occurred. In places where the movement was slow and gradual and could be detected by the convergence measurements, additional supports such as cement grouted rock bolts and invert shotcrete was placed.

In some instances where the failure was sudden such as within a few hours after the deformation is observed and no time is available to provide additional supports vertical and horizontal struts were placed across the tunnel quickly to avoid a major collapse. After arresting the collapse temporarily the deformed supports were removed and a new support system was installed while removing the temporary placed struts. After completing the new support system convergence measurements were taken until the readings showed a steady value for a reasonable time duration. In places where the ground moved fissure grouting was done to stabilize the deformed ground.

At Stn.30+36 on the downstream drive there was a sudden increase in water inflow and the tunnel excavation was suspended and arrangements were made to install additional pumps at the face. However the increase in the flow was excessive to handle by the existing pumps and the drive flooded 240m backwards before the dewatering arrangements were made. The dewatering operation continued for nearly two days before the work resumed at face thereafter.

Eight support systems were developed from the combination of rock bolts, shotcrete with or without wire mesh and steel ribs depending on the ground conditions encountered during excavation. These eight different support systems are shown in fig.4. The ground supports were applied immediately following the excavation and the New Austrian Tunnelling Method (NATM) was used whenever appropriate which consist mainly of rock bolts and shotcrete lining of varying thickness reinforced by several methods such as spile bars and lattice arches with continuous monitoring of deformation measurements.

To deal with large water inflows which was anticipated according to the geological report sump niches were constructed on the left hand side of the tunnel at approximately 500m intervals with the sump being below the tunnel invert. These sumps were provided with two submersible pumps in each having a capacity of 200 litres per second. As the high water inflows were encountered in the upstream drive with a highest initial flow of 350 litres per second carrying silt inundated the rail track and water started flowing over the full width of the invert causing difficulties to the movement of locomotives. To overcome these difficulties water was pumped from the face to the adit junction using pumps until two side drain channels were constructed along the tunnel to handle a maximum flow of 350 litres per second.

Due to the difficult ground conditions encountered the tunnel excavation took 33 months to complete against the 25 months of original programme and downstream drive extended 700 m beyond the anticipated break through chainage due to slow progress in the upstream drive. The support measures during construction had to be increased over 340 percent from the original foreseen quantities, resulting considerable extension of construction time. Several face work suspension due to collapses and dealing with excess water inflows also contributed to the additional time required for excavation work.

Since there was a large discrepancy between the anticipated and actual rock conditions some part of the tunnel lining needed to be reinforced which was not foreseen in the original design. For this purpose the rock modulus of different rock grades were required. To establish a correlation between the rock mass rating encountered from the tunnel logs and the Young's modulus it was decided to carryout the sample tests on different rock grades. Eighteen test hole locations were selected and a refraction survey was done in a ten meter deep bore hole fixing geophones at one meter intervals. From the data obtained by the refraction method and the results of some core samples which were subjected to point load tests a relationship was develop between the Young's modulus and the rock mass rating. This Young's modulus was used to identify the areas where reinforcement were required.

CONCLUSION

It is evident from these three case histories that information of ground condition provided by the bore holes is never complete. Drilling of probe holes from the advance face to explore the ground conditions ahead of the face is a appropriate method. The length of the probe hole is generally selected to suit for one week progress. The flushing materials and rate of penetration is a good indication to predict the ground ahead. It was also helpful to identify water bearing zones. Probe holes act as a drain hole in situations where water present and the effect of water is reduced.

The three case histories discussed above also indicate that when an unexpected ground is encountered it is very important to investigate the extent of the weak zone and to identify possible alternatives so that the decision could be made in the most economical method which automatically reduce both time and cost without abandoning part of the already completed tunnel.

ACKNOWLEDGMENT

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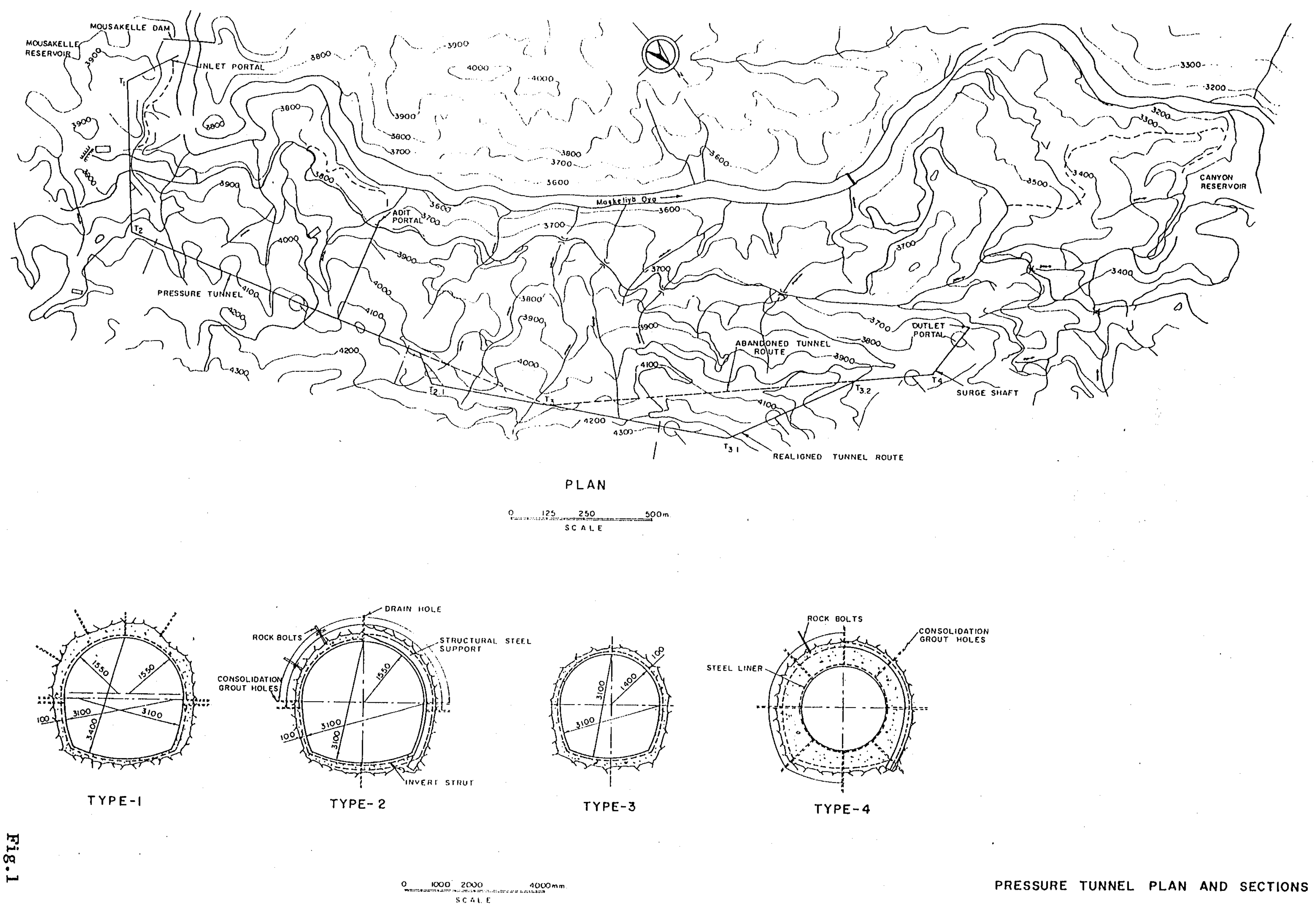
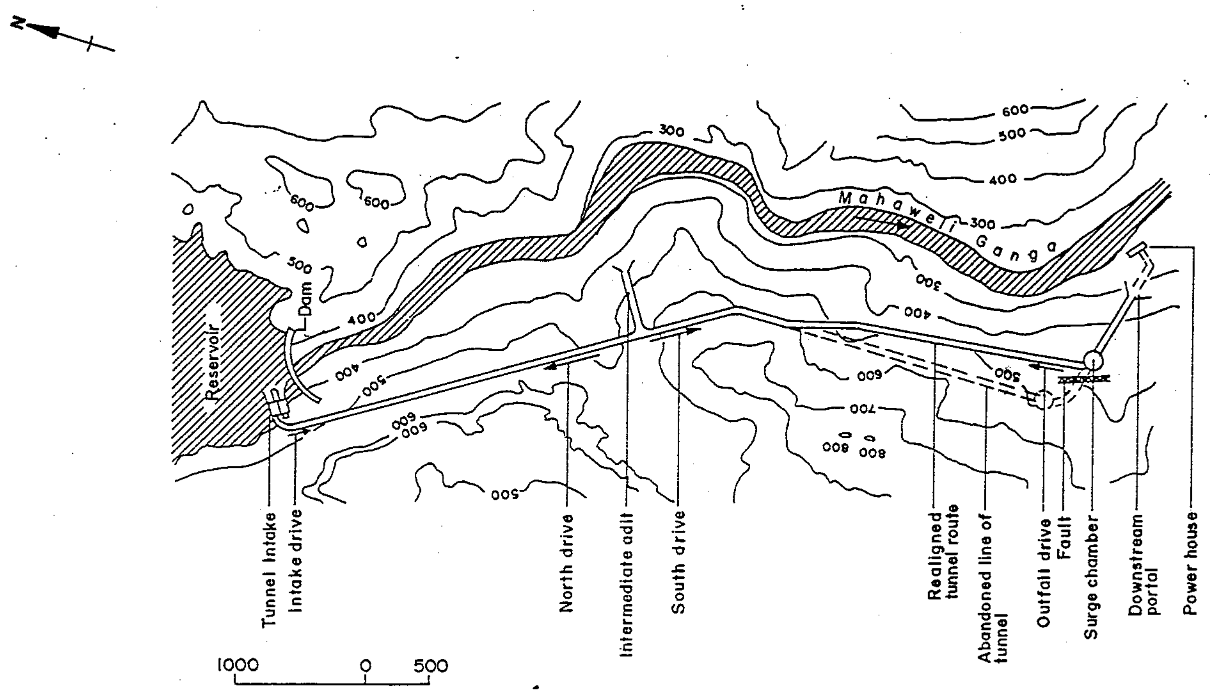
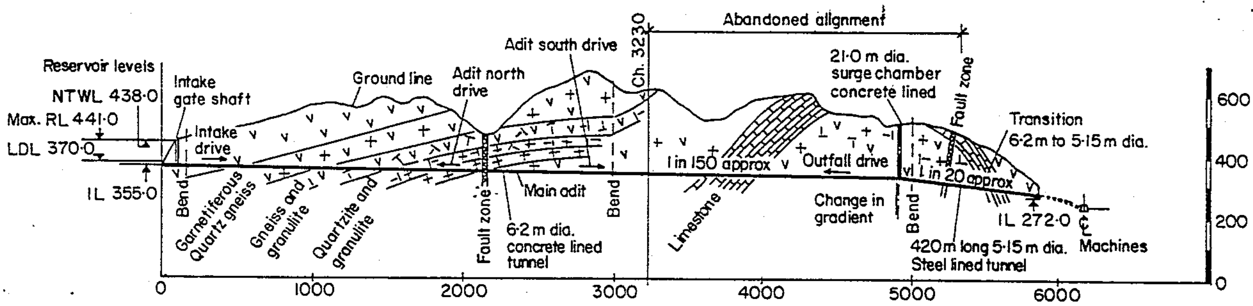


PRESSURE TUNNEL PLAN AND SECTIONS

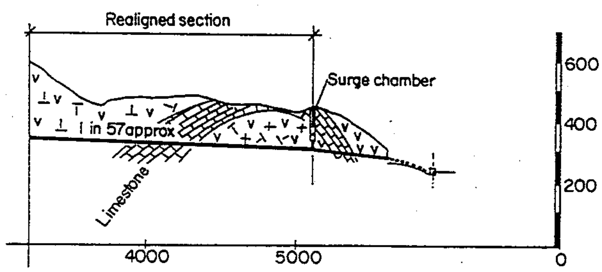
Fig.1



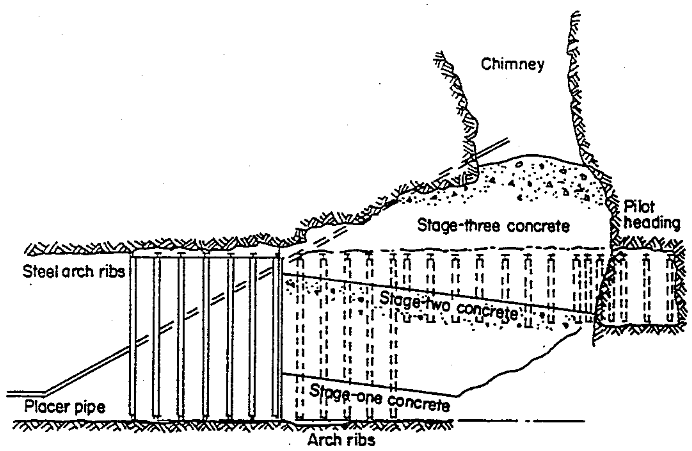
Plan of Power Tunnel



Longitudinal section of tunnel

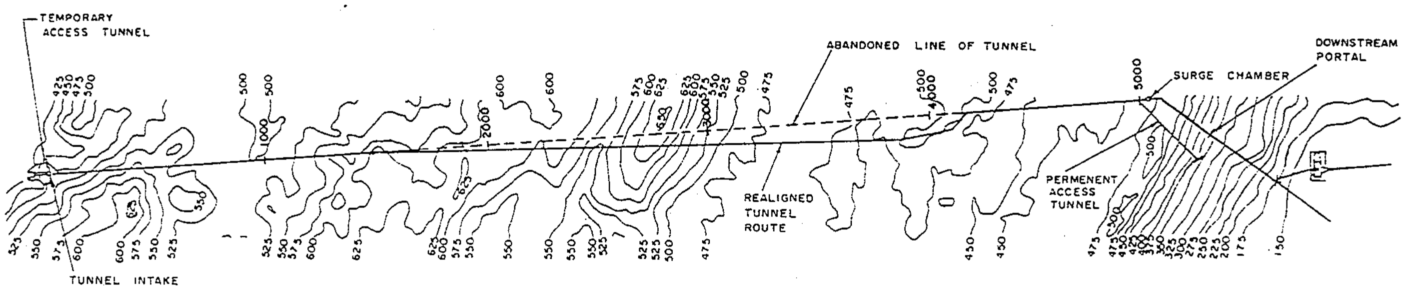


Longitudinal section of tunnel (realigned)

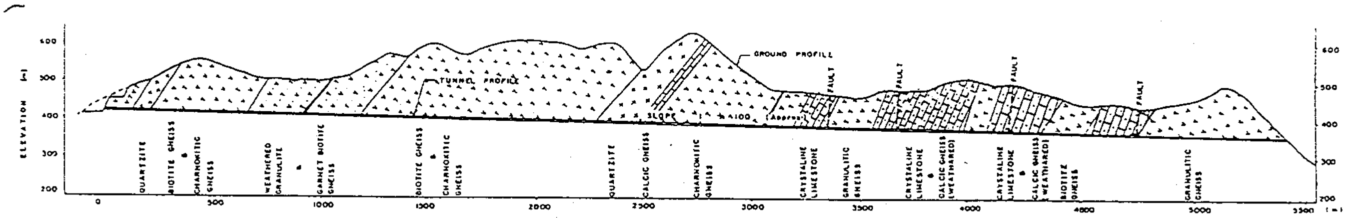


Treatment of rockfall area

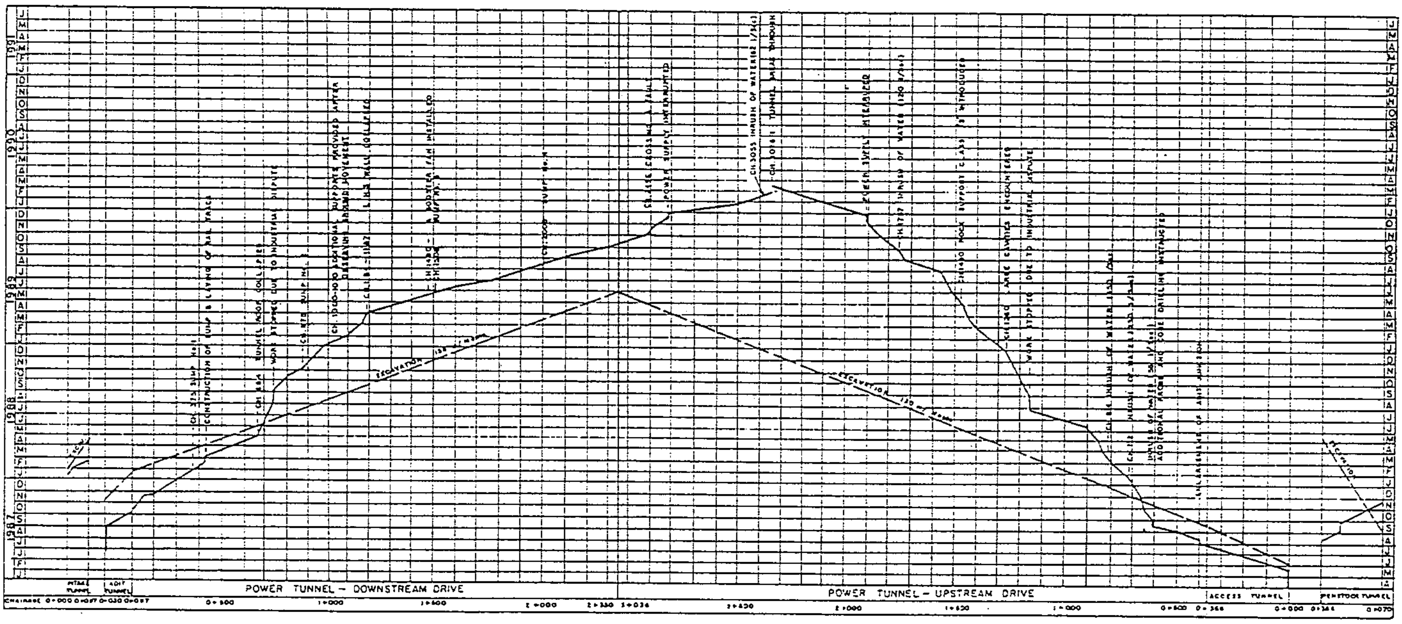
Fig.2



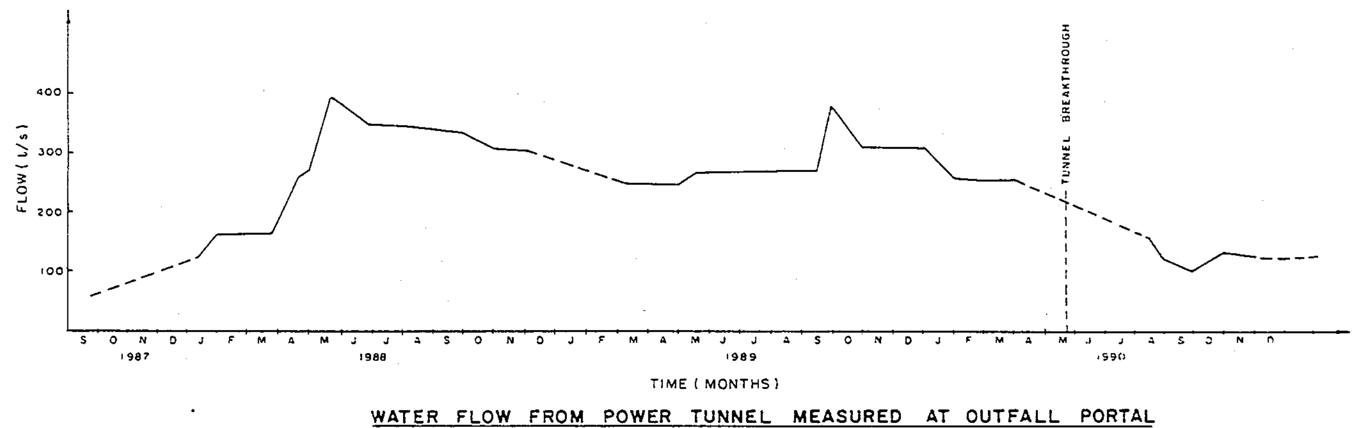
PLAN OF POWER TUNNEL



LONGITUDINAL SECTION ALONG TUNNEL TRACE

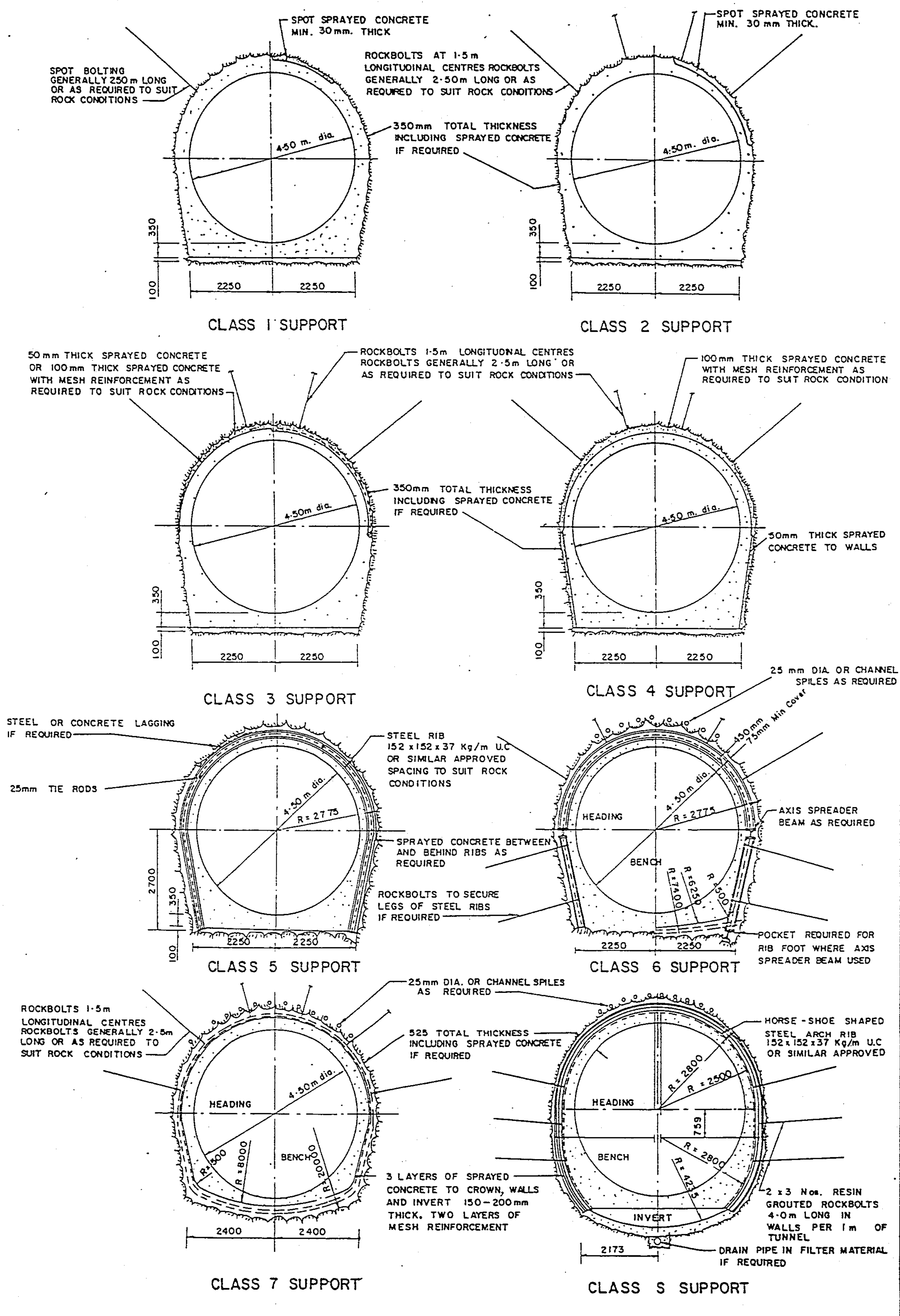


PROGRESS OF TUNNEL EXCAVATION



WATER FLOW FROM POWER TUNNEL MEASURED AT OUTFALL PORTAL

Fig 3



CLASIFICATION OF ROCK SUPPORT CLASSES

Fig.4

GEODETIC SURVEYS IN CONSTRUCTION OF DAMS AND TUNNELS

by T. Somasekaram

Introduction

One of the most fundamental rules taught to the trainee Surveyor is that all measurements are liable to error. There is no such thing as an error free measurement. How good a Surveyor the trainee becomes depends on how well he absorbs this fundamental truth. On the other hand, the Engineer constructing a dam or a tunnel would like to know the exact quantities, length, breadth, height and the 3D coordinates of every point of interest to him. For example, you are excavating the Polgolla tunnel from this two ends, you would like the two cylindrical holes to mesh as perfectly as possible when they meet, far below the top of the hilly surface. It would be disastrous if they failed to meet or even if there was a mismatch of a f*action of a metre. And just to put the problem in perspective, we may note that the vertical at one end of the tunnel is inclined to the vertical at the other end 6 km away by almost 3 minutes of arc.

How do we solve this problem, this challenge, of providing this information required by the construction engineer as accurately as possible is the subject of this paper.

2. Geodetic Surveying

As you of the Geotechnical Society know very well, the word Geodetic is derived from the Latin word Geo meaning the Earth. It refers to the precise and extensive surveys which had to be carried out to determine the shape and size of the Earth, after Newton put forward his theory that the Earth ought to be ellipsoidal in shape as a corollary of his theory of gravitation. In modern usage, geodetic survey means very accurate surveying and includes precise levelling.

Geodetic Surveys have a long and interesting history. Starting with huge theodolites with one foot diameter circles (large circles being easier to sub-divide) and invar tapes (invar having the least coefficient of thermal expansion), large areas of the earth were covered with Geodetic Triangulation networks. The Great European Triangulation, the American Triangulation. The Great European Triangulation, the American Triangulation, The Great African Arc and the Trigonometrical Survey of India may be cited as examples. Mt. Everest is named after Sir George Everest, Surveyor General of India, who completed the Great Trigonometrical Survey of India.

3. The Sri Lanka Geodetic Framework

In Sri Lanka, a base line was measured in Negombo in 1857 - you can see the towers at the two ends along the road parallel to the main road just south of Katunayake airport - and Triangulation covering the south of the country and extending to Batticaloa was connected to the Indian Triangulation by a narrow chain of triangles. A recomputation was done in 1888. Triangulation covering the rest of the country was tied on to this figure at the beginning of this century and extensive Cadastral Surveys carried out to settle people in the Dry Zone produced difficulties in fixing the new work to the old. It was decided to do a complete recomputation in 1929. The two bases at Negombo and Batticaloa were measured using Invar tapes standardised at the National Physical Laboratory in London, astronomical azimuths using stars were observed at the two bases and the entire figure was adjusted by the Least Squares method. The Geodetic co- ordinates and Plane co-ordinates were thereafter computed. As a matter of personal interest, the entire work was done by my teacher in Cambridge, Mr. J.E. Jackson,when he was in charge of the then Training School at Diyatalawa, in 1933. These are the co-ords we have used up to the present. So all our development work, from Minneriya and Gal Oya to the Mahaweli, including all the dams and tunnels, are based on the geodetic co-ords laboriously and painstakingly computed by Mr. Jackson using a manual Brunsviga calculator.

Similarly, we have a good primary level net, observations of which were commenced in 1925. We have systematically kept the level net updated, by re-measuring the lines and replacing the lost benchmarks. The adjustment and computational procedure not only takes care of observational errors, but also the curvature of the earth, so that if you tie your work to a Survey Department benchmark, you are on safe ground.

But as in all other professional fields, times have changed - and changed drastically for us surveyors. In 1948, the Geodimeter, using light waves, was invented in Sweden and the Tellurometer, using radio waves was invented in 1956 in South Africa. So the will-o-the-wisp we Surveyors had been seeking for centuries, of trying to measure distances without having to painfully lay a tape along the line, was at last found. The new methods were both fast and accurate - and "user friendly" if I may use modern jargon. The computer also made a fundamental difference. At Diyatalawa, my teacher Mr. Jackson has laboured for months to carry out his computations. In the 1970's and 80's, my students at Diyatalawa were carrying out Least Squares adjustments and trying out different weighting systems, in hours. Practical necessity also arose. We ran into difficulties in closing some of the EDM traverses between Trig Stations, not due to any errors in the traverse, but because they are more accurate. Work done as part of the training of Asst. Supdts. of Surveys showed that our Triangulation is not of First Order i.e. positional accuracy better than 1/100,000 but of second order, i.e. 1/40,000. In some places, it is worse. So we embarked on a Re-Triangulation programme during my tenure as S.G. in 1992, using the most modern methods and equipment - Wild T3 theodolites, EDM, pre-analysis of the figures by computer. We hope to add a few GPS (Global Positioning System) observations before we release the new figures in 1994.

4. Geodetic Surveys for Dams and Tunnels

As I said at the beginning, Engineers want exact figures for construction of dams and tunnels. The Sri Lanka triangulation framework existing now provides an accuracy of the order of 1/40,000 and that too only on the tops of widely spaced hills, about 40 to 60 km apart. So what we do in practice is to carry out an accurate local geodetic survey and tie it to the national net. So you get co-ords related to the rest of the country but with no local inconsistencies. Typically, the local Triangulation points are established in close proximity to the worksite, in consultation with the Engineers. An actual case will illustrate what is done.

An Example Take the case of the Canyon Power Project in the Maskeliya Oya scheme. The CECB requested the Survey Department to fix 22 points in the vicinity of the proposed tunnel. This was reduced to 10 points in consultation with the Engineers. The work had to be done on an urgent basis and was done in 20 working days, in March/April 1978. A new Trig point Rickarton was established, inside a triangle formed by three old secondary Trig points. See Fig.. After the co-ords of the new Trig point were determined, EDM traverses were run to establish the co-ords of the 10 points.

The total misclosure in each triangle, i e the difference between the observed total and the sum of 180 degrees and the spherical excess, was of the order of 10 seconds and each observed direction received an adjustment of 3-4 seconds. These are not high precision results, but considering that the figure was small with the longer side of about 7 km, was sufficient for the purpose. The angles were observed by Wild T3 on 4 zeroes i.e. different positions of the circle, on both face left and face right positions. This compares with 16 zeroes, on both faces, observations at night and spread over two nights, which are the specifications for truly First Order measurements. In The results achieved at Maskeliya, the uncertainty is of the order of 5-10 cm and must have been acceptable to the Engineers.

5. Dam Monitoring

This fruitful co-operation between the Surveyor and the Engineer does not end with the Surveyor providing the required information for the construction phase. It has become necessary, due to failures both here and abroad, to monitor dams and tunnels after they are constructed. We were invited by the Mahaweli Authority to monitor the dams at Randenigala, Kotmale, Maduru Oya and Rantambe for possible small movements. Well defined points were established on places pointed out by the Engineers and geodetic observations were made and the results analysed by computer for a few years. Now the work is being done by the staff of the Mahaweli Authority. Both the experimental procedure and analysis were designed to detect movements of the order of millimetres. Nothing startling occurred during our period.

It may interest you of the Geotechnical Society if I refer to a unique experiment the Americans are carrying out. The San Andreas fault is a major fault, visible from the air and widening at the rate of about 1-2 cm per year. It is supposed to be the meeting line of the American and Pacific Tectonic plates. Trig points have been established on both sides of the fault and measurements are carried out every day. Infact, they went to the extent to flying a plane with a thermometer and barometer dangling from the plane, along the line of sight between two survey points, as EDM measurements are affected by temperature and pressure and they were not satisfied with measuring these at the two ends, as the rest of the world does. Their interest is that any sudden movement may serve as an early warning of an earthquake in California. It may not be necessary for us in Sri Lanka to do something so drastic, but it may be useful for our two professions to join together to monitor some of the reported faults, particularly if they are across dams, at least once a year.

6. Deformation of the Earth's Crust

Since the theme of you seminar at which I am presenting this paper is Geotechnical Engineering in River Basin Development, it may be appropriate for you to look into what happens to the plastic crust of the Earth when you impound a large mass of water. I studied this from a Surveying point of view and a student of mine S.D.J.P. Dampegama and I presented a paper at the Annual sessions of the SLAAS some years ago. We computed the effect on the Geoid and the Direction of the Vertical caused by impounding about 2.5 billion tonnes of water in the Randenigala, Victoria, Kotmale, Ulhitiya and Maduru Oya reservoirs. Our computations showed that the Geoid changed by about one (1) mm in the vicinity of the reservoirs and the change ion the direction of the vertical was of the order of 0.01 second. Both are not significant numbers. But we did notice an amazing coincidence. Practically all the recent landslides have been inside the are of maximum change. In the South African case, where the Hendrik Verwoerd dam impounds 5.0 billion tonnes of water, the changes in the Geoid were of the order of 2-8 mm but the actual crustal deformation was 30 mm. Your society may perhaps study this effect and also try to see whether there is any casual link between the large reservoirs and the earthslides.

7. Conclusion

The construction of dams and tunnels and their subsequent monitoring and study, calls for the highest level of expertise in both the Surveying and Engineering professions. We have to perform at our highest professional levels to achieve good results. I am grateful to the Geotechnical Society for inviting me to present this paper.

GUIDE LINES FOR DESIGN OF PRESSURE TUNNELS
Dr. G. P. Rajapakse

1.0 INTRODUCTION

The earliest tunnels were those evolved millions of years ago by contortions of the earth's crust, leaving vast underground caverns linked by passages of varying length and size to form underground ways often extending for miles. Again through countless centuries underground rivers have worn their way through rocks deep below the surface. In this way were created some of the outstanding wonders of the world in the form of caves grotesquely ornamented with stalacktites and stalagmites & figures weirdly carved from the solid rock. Similar caves were formed by the action of sea and these and cavern Islands served mankind for many generations as habitations and places of refuge. However, such caves are not tunnels as we understand the word today as they led no where. Thousand of years were to pass before mankind found it is necessary to push far wall of his cave to meet the daylight beyond.

Tunnels constitute some of the most outstanding and courageous feats of engineering in the history of the world, despite the fact that they are hidden out of sight below the ground and cannot arouse admiration. Some of the worlds greatest engineering projects could not have come into existence without tunnels. Many essential railways would have come to a halt before some massive mountain range. There wouldn't have been no great hydro electric schemes, if there were no tunnels to transfer the vast amount of energy stored in rivers, lakes and reservoirs. Tunnels have contributed to the growth of great cities by providing means of communications.
Broadly speaking present day tunnels serve the following purposes:

- Route for transportation (railways, motorised or even waterborne)
- Conveyance of water (hydro electricity, irrigation or drainage, wastewater disposal, water supply)
- Occasionally as route for services - (electric or telephone cables gas pipe lines) and conveyance of air or gas from water balanced air or gas storage in excavated caverns.

In order to achieve the intended purpose and /or to satisfy the design requirements tunnels are constructed at shallow depths (soft ground tunnels) and at depths (hard rock tunnels) with in the ground. Soft ground tunnels are mostly used as route for transportation and major part of them runs through the developed areas.Hardrock tunnel is characterised by the ability of the roof of the tunnel to stand up without special support for extended period of time. Soft ground tunnels on the contrary requires to be supported soon after excavation (generally within 12 hours) to prevent collapse of the ground. The stand up time (as it is called)is also influenced by the distance which the material is required to span in the crown of the tunnel. The term softground includes weathered rocks, alluvial deposits such as silt and gravels, younger (in geological age) weak rocks and faulted old rocks. Generally all these materials are open textured and therefore have high hydraulic conductivity. These tunnels will not be dealt with in this paper as it is outside the scope of the theme of this Seminar. Hard rock tunnels are mostly used for conveyance of water. Major part of it runs through the rock. Except for hydro electricity (power) tunnels in this category are designed as free flow tunnels to achieve maximum flow conditions and economy. Power tunnel flows full and under internal pressure. This paper is mostly confined to design of Power (pressure) tunnels and the design process is adaptable to free flow tunnels as well.

Power tunnels are required to convey water from the intake to a Powerhouse, which may be either surface or underground. Such tunnels may begin in soil or rock and often pass through variety of materials and geologic conditions before reaching the powerhouse. Their prime responsibility is to convey the water safely throughout the life of the project, without detrimental effect on the surroundings. Such affects may include excessive leakage from the tunnel, instability of surface soil or rock resulting from the seepage, saturation and softening of agricultural land, and pollution of ground water and surface streams due to organic content of conveying water. The effects canbe controlled by careful positioning of the tunnel, and by selection of the appropriate lining and treatment for the various parts of the tunnel.It is esenential to understand the geologic conditions along the tunnel route, it's behaviour relative to the hydraulic forces that will be applied during operation. For this, geologic and geotechnical conditions could be defined with modern investigation techniques and tests. Appropriate materials are available to line the tunnel and to treat the surrounding rock. However, it is necessary that the conditions be investigated and that designs established by those who are experienced in Engineering geology and Applied rock mechanics.

2.0 DESIGN PHILOSOPHY

In the design of a structure, designer has to ensure that:

(a) the best economic return on capital invested

(b) the structure does not fail under imposed loadings.

We commonly think of tunnel failure in terms of collapse of tunnel lining or surrounding ground. Failure might include unacceptable displacement of the tunnel surround (lined or unlined), inundation of the tunnel by water or conversely loss of water in a water conveying tunnel. Encountering of unforseen ground conditions during construction, leading to delays, additional cost is also 'failure' in a different sense. It is towards these combined requirements of minimising and avoidance of collapse failure both during and post construction periods that much design effort has to be directed. If a design can satisfy these criteria the other types of failure in most cases be automatically avoided and the cost incurred will fall in line with predictions and budget.As the likelihood of collapse is almost entirely a function of the ground or rock conditions surrounding the tunnel with the level of risk affected by such factors as excavation size, the presence of ground water etc. Ground conditions are therefore far reaching in effect and a knowledge of them is of paramount importance to the tunnel designer. The design for the tunnel should satisfy as far as possible the following objectives -

- to minimise Construction cost, construction time in relation to cost, to minimise disturbance to others and to simplify the construction in the short term.
- to minimise maintenance cost, to maximise capital return, to avoid deterioration and collapse of the tunnel, to provide satisfactory environment for the intended purpose in the long term.

3.0 DESIGN PROCESS

"Many difference aspects, disciplines and people are involved in the design process (for eg. geology,hydrogeology, geomorphology, rock mechanics, geophysics, planning, design, construction) and they must all be drawn together in a consistent and compatible way to produce the required result"(Gosschalk, 1985). Mr.Goschalk in his paper has summarised the design process in which the tunnel designer should be responsible throughout as follows:

1. Collection and review of available data
2. Reconnaissance of site
3. Arrangements to obtain additional data needed (site investigations)
4. Conceptual design
5. Analysis of data
6. Detailed design
7. Arrangements for construction (ensuring construction methods meet design requirements)
8. Supervision of construction
9. Monitoring of performance (ensuring it meets design assumptions)
10. Modifying design if necessary (to suit actual conditions found)
11. Operation and maintenance instructions (to confirm with design assumptions)

The basic problem designer faced with is the non availability of adequate time and precise data necessary for refining theoretical analysis. At the depths underground common in tunnelling, it is too expensive and impossible to get the comprehensive data needed on rock conditions until the tunnel excavation reached that point in question. The basic design needs to be decided before then or dangerous conditions and expensive delays may result. The basic design is to be based on existing data and additional data acquired subsequently. Having appraised the available data, the additional data required can be assessed and steps must be taken to get it. Table 1. indicate the relevant geothechnical data and the site investigation appropriate in each case (Goschalk 1985). This must be a well planned and well executed thorough site investigation.

4.0 CONCEPTUAL DESIGN

Design of a tunnel falls into two quite separate and almost unrelated parts

- The shape and size i.e. the profile.
- The alignment of the tunnel, arguably this is the more important one.

4.1 TUNNEL SHAPE AND SIZE

The intended purpose of the tunnel will have an overriding influence on the design decisions affecting the size and shape of the tunnel and form of lining. The basic and essential problem which should be considered in the design of tunnel is to eliminate tangential tensile stresses which increases the width of joints or originates new cracks and to minimise tangential compressive stresses which causes excessive deformation around the boundary of excavated area. Most tunnels have circular or nearly semi-elliptical cross sections. The conventional shape of tunnel is based on the assumption that the vertical stresses are more than the horizontal stresses in the rock. Terzaghi and Richart (1952) proposed that in geologically undisturbed regions in sedimentary rock the horizontal stress depends on the magnitude of the vertical stress and Poisson's ratio. For a Poisson's ratio of 0.25, value of horizontal stress is(approximately) one third of vertical stress. Based on this assumption a suitable geometrical shape is either circular or semi elliptical with major axis in vertical direction.

Prof.Amirsolyemani (1988) has shown that when the horizontal stress is several times the vertical stresses at a point in rock (highly stressed) the geometrical shapes of cross section be based not only on the orientation and location of joints or discontinuities, but also on the basis of direction and magnitude of main principle stresses within the rock mass. By this method it is possible to reduce extensively the compressive and tensile stresses around the perimeter of excavation.For best strength characteristics and to reduce undesirable tensions in rock and concrete, circular, elliptical , horse shoe and D shapes for tunnels come out best in that order. However, for ease of access during tunnelling operations except when tunnel boring machine(TBM) is to be used, selection will be in the opposite order. If ground conditions are known to be poor, circular section should be used. Horse shoe shape which is commonly adopted compromise between the conflicting requirements and uncertainties as to ground conditions.

The minimum size of a long tunnel that can be economically be driven is about 3 meters in height and width or in diameter. Hence, even if it is satisfactory from an operational view point to have a smaller size, it is not advisable to specify a smaller size because it is a factor directly effecting the cost of construction of the tunnel. The maximum size that can be constructed full face is about 10 m(30ft). provided that ground conditions permit such a size in terms of stability. For larger tunnels top heading and bench excavation have to be adopted which adds to cost and schedule. For water conveying tunnels the maximum size should be such as not to exceed the allowable velocities to avoid damage to the rock or lining. These are of the order of

- unlined water ways
    - for unpaved inverts 1.5 m/sec.
    - for paved inverts 2.75 m/sec.
- lined water ways
    - for concrete 4.0 m/sec
    - for steel 5.0 m/sec

In the case of power tunnels the tunnel size depends on an economic study to evaluate the marginal cost of increasing the diameter(size) so as not to exceed the marginal present value of the energy which will be saved by this increase due to the reduction in head losses due to friction along the tunnel.

For a tunnel where good rock conditions are likely, it may be advisable to plan excavation to a size which can be left unlined. Then if the rock conditions encountered is worse than expected lining can be provided and head losses will be relatively low. If rock conditions are good enough the tunnel can be left unlined. Further, thickness of lining has to be catered for in deciding the excavated profile of the tunnel.

4.2 TUNNEL ALIGNMENT

Selection of alignment is less influenced by the specific purpose of the tunnel and is governed primarily by economics. The projects layout is primarily determined by the topography of the regions. The selection of best combination for maximum Power generation and location for the best dam sites is based on topography. However the precise location of the dam site will depend on the geology of the area. Then the tunnel alignment has to fit rest of the Scheme. This will be the shortest route between the dam and outlet. This route has to be the one which will present the least difficulties to progress during tunnelling which will permit completion of the project on schedule and within the available budget. This is governed by the ground conditions - geology along the tunnel alignment which normally take second place to geology of dam site.

4.2.1 TUNNEL ALIGNMENT -PRIMARY FACTORS

In establishing the optimum alignment of tunnel number of factors must be taken into account. The primary factors which are likely to have a dominant role in overall economic include

- Difficult geological condition
- Schedule for project completion
- Available contractors and equipment

These factors impact strongly on the final cost when they arise during the course of construction the result is costly overruns or project delays.

Difficult geological conditions

Having completed the basic design of the tunnel based on available data and additional data acquired the route chosen for a tunnel will be detailed on a plan and longitudinal section along the tunnel can be drawn. The possibility of difficult geological conditions must be carefully assessed and avoided if possible, by selecting an alternate route even if at first glance it appear more costly. Determination of these conditions can often be used with preliminary evaluation by engineering geologists or engineers experienced in difficult tunnelling. In some cases such an evaluation may require drilling or other field investigations before even a preliminary geologic assessment can be properly undertaken.

Interpretations of stereo photos is the best way to discover location of surface evidence such as faults or fissures (lineaments) which crosses the possible route which can then be examined on the ground. Geological reconnaissance or surveys will define the orientation of structures at the surface from which formation and structures of the rock and hence its likely nature at depth could be interpreted. One of the most important factors which determines the stability of the rock around underground openings is the orientation of joints. In general it is best to avoid aligning a tunnel parallel to the planes of main joint system or faults or fissures because they usually result in slabs in the roof and walls with little interlock with other slabs. The route selected should be such that portals at each end be located in stable hillside with rock cover over the crown be more than one tunnel diameter.It may be necessary to adjust the tunnel alignment to pass beneath suitable high ground with adequate good rock condition above the tunnel for locating the Surge Chamber or to allow easier access.

Schedule for Project Completion

Many power tunnels are significantly longer and may take a very long time to complete. The schedule can be improved by providing additional access adits, by specifying a certain manner of construction or by ensuring that majority of the tunnel is driven in sound rock via a longer route where progress will be rapid and reduced support and lining are required.

Available contractors and equipments.

The availability of experienced contractors and modern equipment can be a major concern if the client is not willing to risk the use of high speed TBM. Standard drill and blast method have to be used which will result in longer construction schedule. Appropriate access adits, a longer construction schedule or high speed drill and blasting method may have to be specified. Normally, the specified slope is 0.5% for a tunnel (for gravity drainage) but if locomotives haulage is planned the maximum slope is limited to 2% and for pneumatic tyred equipment 10-12% slope is specified. Steep grade tunnels(shafts)slopes should be more than 50 degrees to the horizontal and allow for self mucking.

4.2.2 TUNNEL ALIGNMENT -SECONDARY FACTORS

Amongst the many secondary factors that impact on the selection of the tunnel alignment, the most common and most important ones are:

(a) Hydraulic grade
(b) Prevention of Hydraulic Jacking or Uplift.
(c) Leakage from pressure tunnel
(d) Stability of adjacent slopes
(e) Selection of final lining
(f) Stability of the tunnel
(g) Rock support-temporary and final
(h) Steel lined length
(i) Grouting

(a) HYDRAULIC GRADE

The tunnel must be maintained below the hydraulic grade line for all modes of Power plant operation, including hydraulic transients to prevent sub atmospheric pressures in the tunnel.This requires evaluation of head losses by friction along the tunnel which is a function of type of lining or in the case of unlined tunnel the roughness of rock walls, size of the tunnel and velocity. Head losses may be determined by the Darcy Weisbach formula or the Empirical Manning equation. These are given in Appendix I

For the purpose of linking the intake to the powerhouse there are innumerable alignments which may be chosen. Technical and economic analysis must choose among factors such as high versus low level alignments, the use of shafts versus inclined sections, all satisfying the primary requirement of positioning the tunnel to ensure submergence beneath hydraulic grade line. Fig.1, shows a number of variations that are commonly used. Each of these arrangements have advantages and disadvantages that must be compared before the most economic one is selected. Some of the factors which are often not given due considerations are:-

- Construction access

Inadequacy of the No. of adits required to meet the overall schedules especially when the tunnel driving rates are not as assumed. Adding extra adits during constructions may disrupt the basic designs and add contractual problems and costs. It also requires that they are suitably plugged for operating. Intermediate access adits may also be necessary to handle suspected delays due to poor geology, whose details cannot be obtained in advance.

- Access for steel liner installation.

This requirement may be overlooked specially for underground powerhouse. Installation of steel liners should be planned as a separate operation with independent access.

- Prevention of hydraulic Jacking

The entire tunnel must be set deeply with in the rock mass to ensure that adequate insitu compressive stress is available to prevent hydraulic jacking.

- Selection of temporary support and final lining, handling the drainage and ventilation during construction should also be considered in establishing the position and alignment of tunnel.

(b) PREVENTION OF HYDRAULIC JACKING OR UPLIFT

Adequate confinement refers to the ability of the rock mass to withstand the internal pressures from an unlined waterway. If the confinement is inadequate hydraulic jacking may occur. This effect can be developed if hydraulic pressure imposed within a jacking surface such as a joint or bedding plane exceeds the total normal stress acting across the (potential) jacking surface. It will occur in any direction where movement of rock masses can develop due to a lack of adequate compressive insitu stresses. Thus vertical lifting of horizontally bedded rock, jacking or hoisting of rock masses towards valley walls,jacking of rock blocks into adjacent underground openings or opening of fractures in a compressible rock mass and existing joints can occur.

This may result in jacking of a large mass of rock away from the tunnel with excessive leakage and large scale landslide or instability and even flooding of underground power houses. When such failures occur, it often takes many months to diagnose the problem and complete repairs. The financial losses will be heavy with considerable distress to inhabitants due to power shortage. Therefore, a careful and conservative design to prevent problem of this nature must be adapted. When compared with project delays and lost revenue, the cost of actual repairs, and the mental anguish of having to redo an unsafe design, the cost of using necessary additional steel liner or reinforced concrete liner (not necessarily an appropriate solution always) to ensure a conservative design is well justified.

For many years unlined tunnels were located by ensuring that the weight of the rock, and in some cases the soil, vertically above the tunnel was at least equal to the static water head. This vertical rock cover criterion is appropriate for relatively flat topography. Many failures have occurred due to a deficiency in vertical cover criterion. However, where the power tunnel approached valleys and either frontal or lateral cover was low or where geologic conditions resulted in low stresses, failures have also occurred.

Once the problem of low stresses was recognized, steps were taken to either measure the rock stresses, or to estimate the probable stress levels by stress analysis. Rock stress measurement could be done either by over-coring or by Hydraulic fracturing test. Photo elasticity and finite element method could be adapted to assess stress conditions around valleys. Hydraulic fracturing test conducted in drill holes is principally used to obtain a measure of the minimum principal stress in the rock mass and can also be used to measure the three dimensional stress field. This test simulates the actual effects that will be imposed on the rock by the tunnel water. It is important that drill holes used for this test crosses all joint systems, especially through going or master joints, and those which are sub parallel to valley walls. All these tests must be performed and interpreted carefully to ensure that insitu stresses of the rock mass is being measured and not some characteristics of the pumping system.

Based on the results, designer can rationally position the tunnel or design a suitable lining to resist hydraulic forces if the tunnel cannot be safely located. Other designs methods, such as grouting, and pressure relief and drainage, can be used to limit seepage pressure and thereby prevent Hydraulic jacking. Such methods are not considered to be a safe approach and therefore cannot be recommended. This may be used where potential failures can be tolerated or when a problem has arisen that cannot reasonably be solved by a more direct approach. However, such measures must always be taken in conjunction with sound geological interpretations, to ensure that the design measures are compatible with the behaviour of the various geological materials when subjected to high pressure seepage water. Material boundaries,probable stress, permeabilities and deformability (modulus) must be determined by appropriate geologic and testing methods.Hydraulic jacking can only be prevented by steel liners that accepts the full internal pressure without contribution from the rock.

In the event that data on insitu rock stresses are not available, simple design rules can be followed that will ensure safety. For a pressure tunnel positioned beneath a near horizontal surface vertical rock cover criterion for confinement (see Fig.2.) may be adequate subjected to the conditions that insitu stress measurements will be made during construction. Due to the differences in densities of soil and rock it will be necessary to calculate the overlying weight separately.This is significant in areas of deep tropical weathering.In the absence of soil cover or when it is removed by land slides only the rock cover is adapted in calculations.

A factor of safety of 1.3 is adequate against up lift on horizontal planes such as bedding planes or joint combinations. In the event that the lateral stresses are lower than the vertical,consideration must be given to hydraulic jacking against vertical planes(joints or faults)particularly where deformable rock exist. In steeper topography the side cover may govern,in that it can be less effective than the vertical cover.In view of this the criterion shown in Fig.3 was developed for

the Snowy Mountains project in Australia. Norwegians based on
their experience developed the criterion shown in Fig.4.It is the minimum rock cover(minimum distance from the tunnel to the rock surface) C.rm ie used. The varying slope angle of the valley side is included in this criterion. Fig.5. is a comparison of the minimum cover ratio C.rm/H.s obtained when using the above criteria for slope angles from 0 to 70 degrees. It is evident that, the Snowy Mountains and Norwegians criteria are in fairly close agreement and show increasing minimum cover with increasing slope angles. On the other hand vertical criterion yields decreasing minimum cover with increasing slope angles, and at steep slope angles it becomes increasingly incompatible with the project performance observed. For preliminary layout in terms of minimum requirements the Snowy Mountains and Nowergians criteria are very useful tools.

For pressure tunnels positioned near slopes or valley walls, detail consideration of the stress environment is necessary. Particularly in ridges, protruding rock noses stresses may be relieved and therefore give reduced confinement. This is magnified in the upper portions of the above, where as at the base stress intensification commonly exist. Broch(1984) recommended that the topography be diagrammatically corrected to match the overall topographic contours of the surrounding landscape when drawing section for the purpose of assessing minimum cover. This procedure must be done both longitudinally and laterally to account for irregular topography beside the tunnel. This method is as illustrated in Fig.6. Even with adoption of these apparently conservative rules,hydraulic jacking test should be conducted at the critical points along the tunnel.

(c) LEAKAGE FROM PRESSURE TUNNELS

The consequences of excessive leakage from Pressure tunnels are:

- Loss of generated revenue
- Loss of revenue from dependent industries
- Cost of pumping, if the leakage water is collected in sump in underground power stations
- Operation, maintenance, and aesthetic problems when leakage enters the P.H. area
- Development of high hydraulic pressures behind low relative permeability features within the rock mass leading to valley side instability.
- Development of high hydraulic pressure around steel lined concrete encased sections leading to buckling of the steel lining.
- Spring formation above erodible materials in valley sides leading to mud slides.
- Tunnel or shaft instability through piping of erodible, or leaching of dissolvable materials.(Brekke & Ripley,1987)

Excessive leakage can occur from pressure tunnels in two ways. Firstly by hydraulic jacking and secondly through permeable geologic features and the internal pressure exceeds the external ground water pressure despite adequate confinement. Method of dealing with hydraulic jacking have been discussed previously. The problem of permeable geologic features is more difficult to deal with because there are many possibilities for leakage paths. Locations within the unlined pressurised waterways which may be particularly leaking include areas without adequate confinement, areas of local instability area with abrupt changes in cross sections and flow where thrusts develop, and intersections with high permeably leakage paths. Leakage paths are commonly associated with open joints, broken dikes crushed rock in shear zones or adjacent to faults, and permeable beds. Less common but equally important leakage paths include solution channels, the excavated disturbed zone of rock parallel to the waterway and poorly grouted drill holes. These paths will usually dominate leakage in terms of flow quantities and directions, and only infrequently will leakage occur uniformly.

Leakage assessment involves observations measurement and interpretation of geological and hydrogeological conditions of the surrounding rock mass during investigations, construction and initial operation. During investigations information on the natural ground water regime can be obtained and used in combination with information on the geological conditions to develop leakage patterns. Surface springs if any should be mapped and then flow quantities monitored. Bore holes can be used for geological interpretations, for pressure testing relative to permeability and hydraulic jacking. Piezometer can be installed in some bore holes in order to monitor pressure during construction and operations.

During construction in addition to geological mapping and identification of potential leakage paths leakage is to the openings (Tunnel) can be mapped and flow quantities monitored in a manner consistent with observations of surface springs and piezometric levels. This information could be used to estimate rock mass permeabilities and potential leakage during operations. However, the leakage potential may well be underestimated if the confinement or rock cover is inadequate, or if leakage paths contain soluble materials. Also, for sites where leakage is predominantly along joints the rock mass permeability will likely to be lower during construction than during operation because of lowering of the water table, an increase in effective stress across the joints and an accompanying reduction in joint aperture. During initial filling (critical period) and long term operations, leakage rates piezometric levels and surface seepage should be monitored to detect changes in the leakage conditions. Some leakage (seepage) loss may be allowed depending upon economics and the probable effect of seepage on the stability of the terrain and its effect on the environment. If possible, the tunnel should be placed such that the hydraulic grade line is below the permanent ground water table which is evaluated by experienced geologist or Hydrogeologists based on the data collected (referred to) earlier. The most common impervious lining used for leakage control is steel with a concrete or concrete/grout surround. A composite liner can be utilized where the steel is thin and an inside layer of concrete provide buckling resistance against external pressure. Other types of impervious lining which have been used are bituminous coated copper, sprayed rubber, and plastic normally with a concrete liner placed inside to cater for buckling. However, very little performance data is available on them and the design and construction is complicated. Concrete and shotcrete applied without reinforcement must be considered as pervious, when reinforced will act as semipervious lining utilizing enough reinforcements to distribute and control cracks to a specific width. With carefully controlled techniques and pressures, grouting can be used to reduce permeability around a reinforced concrete liner.

(d) STABILITY OF ADJACENT SLOPES

An important consideration in assessing the consequences of leakage from pressure tunnels and shafts is the stability of surface deposits on adjacent slopes. Of particular interest are the relative permeability of the surface deposits and the rockmass in terms of pressure induced failure, as well as susceptibility to flow induced mud debris slides. Attention needs to be paid not only to slopes in the immediate vicinity of the pressure tunnels and shafts but also to slopes in the more general area that may be susceptible to failures because of connected leakage paths. When the surface deposit are of lower permeability than the leakage paths in the rock mass leading to them, water pressure will develop behind the deposits, so that susceptibility to sliding will be increased. It should be noted that high water pressures can develop behind low permeability features within the rock mass and can lead to deep seated failures. Stability of the slopes should be established under these conditions with adequate investigations and studies.

(e) SELECTION OF FINAL LINING

For a pressure tunnel selection of final lining is a process which begins in the design stage but does not end until construction is complete and the geological conditions are known in detail. Preliminary liner design may be modified to suit actual conditions observed in the tunnel during construction. This aspect has to be catered for by providing flexible specification and contract document which allow design modification without unfair penalty to contractor and the client.

There is divergence of opinion on the use of lining amongst designers. Some favour completely lined tunnels on the basis of ensuring long term performance without the need for any maintenance work during operation. Others have championed the advantages of unlined or partially lined tunnels with acceptance of local fall outs, provided they do not prejudice operation. Between these limits there are many options which attempt to optimise cost and performance, but common guide lines for selection of the final liner do not exist.
Basically three factors bear on the selection of the liner.

- Achieving acceptable head loss in the tunnel
- Prevention of excessive leakage either by seepage or hydraulic jacking.
- Ensuring long term stability during filling operations and dewatering.

Headloss

Head loss through a tunnel is principally a function of the wall roughness, the diameter and the velocity. As a result hydraulic equivalence can be obtained between a larger diameter unlined tunnel versus smaller lined tunnels of greater hydraulic efficiency. With small tunnels (3 m) in dia there is a great need for a smooth lining to maintain acceptable head losses. However as the diameter is increased, the wall roughness has less effect on head loss and equivalence is achieved with small diameteral changes. These factors must be assessed in terms of excavation and support cost for both the temporary and final lining and the schedule advantages of unlined tunnel,when the host rock permits and unlined tunnel. In addition the advantages of TBM which can achieve a smoothness almost equivalent to concrete must also be considered. Selection of the most suitable lining for aspect of acceptable head loss is then a complex but standard matter of project economics.

Leakage control

Factors which control the leakages have been discussed in a previous section.

(f) STABILITY OF THE TUNNEL

The final lining selected must ensure adequate stability of the tunnel through out the project life i.e. fall of rock or shortcrete or concrete should not occur. Such an approach requires continuous concrete liner or very high quality rock with extensive rock bolting and shotcrete over an unlined tunnel. On the other hand less support and lining with minor falls in the tunnel such that they do not hamper operation or cause a significant energy loss considered acceptable. The difference in cost and construction schedule between these two design approaches can be very great and significant economic benefit are possible by selecting the approach which allows for minimum rock fall, with periodic inspection and maintenance. There are many design approaches possible, out of these the best economic benefit could be achieved by selecting the approach which allows for minor rock falls and periodic inspection and maintenance. To ensure stability the lining designer must include consideration of the following:

- erosion of rock or joint filling by water flowing under pressure.
- rock support, temporary and final
- hydraulic pressure during watering and operation, dewatering

Erosion

Velocity of water in unlined tunnel does not normally exceed 3 m per second and can cause a progressive erosion of weak rock, resulting in rock fall which can reduce the capacity of the tunnel or in the extreme cases entirely block the flow. The erosion may occur in soft rock, shear or fault zones, or in blocky rock containing clayey or silty seams and veins. Apart from erosion during the operation, materials from these zones can be piped in to the tunnel through cracks, joints or drains holes during dewatering when external pressures around the tunnel are high. The location requiring protection must be identified during excavation by experience engineers or geologist and should be treated during construction.

(g) ROCK SUPPORT-TEMPORARY AND FINAL

Rock reinforcement is required on a temporary basis during tunnel driving and or permanent support during operation of the power tunnel. The degree and the type of support for these conditions can be similar, specially for hard massive rock where no final lining is required, and where rock bolts and shotcrete are required for both temporary and final support. Alternatively in weak, heavily cracked or highly erodible rock, the temporary support required for tunnel driving may be modest, but extensive support or lining may be necessary for operation. Thus, it may be necessary to follow excavation and temporary support with subsequent additional support, up to a complete concrete lining. Invariably the temporary and final support requirements are established by experience during driving of the tunnel. The general support requirement, however, must be determined during design so that appropriate specification and construction methodology can be adopted, that will ensure an economic project, and a minimum construction schedule.

Various classification systems for assessing temporary support exist. In 1946 Terzaghi developed a rock classification system for estimating the loads to be supported by steel sets in the tunnel. Since then a number of other classification system have been developed on the basis of numerous case histories. Some of the more common classification system include:

- Terzaghi's rock load classification for steel arch supported tunnels 1946.
- Deere's RQD and Merrit's method 1972
- Wickham's rock structure rating 1972.
- Barton's Norwegian Geotechnical Institute (NGI) Index classification 1974.
- Bienawski's CSIR method 1976

These systems are helpful in both design and construction planning but do not address all variables, no are they uniformly applicable to all types of rock and various conditions of rock quality. Thus, application must be based on good judgement and experience with various geologic condition.

Following are some specific design points for unlined, shotcreted and concrete lined tunnels. Provided the rock is hard and durable, and not susceptible to solutioning, the tunnel can remain largely or fully unlined. Special zones or areas of weakened rock can be treated with grouted rock bolts and shotcrete, requirements can be estimated utilizing the classification system previously discussed.

Shotcrete is an effective way of improving the stability of tunnels, used in conjunction with rock bolts or passive grouted anchors as in new Austrian tunneling method (NATM), a highly adoptable support system can be developed to meet conditions observed in the field during tunnel driving. With the addition of fibre - reinforcement and silica fume the strength and deformability can be varied to meet special requirements. Shotcrete can be effectively used in most rock conditions. However, for clay stone or weak poorly cemented rock,shotcrete may not adhere properly or may fail easily at the contact. For such conditions, interaction with the rock achieved with mesh reinforced shotcrete and pins is not necessarily achieved with fibre reinforced shotcrete. Thus, selection of the most effective shotcrete method is necessary.

Concrete lining represent the most effective method of ensuring stability while achieving a hydraulically efficient tunnel due to the smoothness of the surface. Construction methods are available that can provide homogeneous concrete with minor imperfection such as cold joints, thermal cracking and honey combing. Even with good mix design and good construction techniques because of such imperfections which are inevitable and because of the variable deformability of most rock, lining cannot be considered as impervious.

Due to associated cost and increased construction schedule, selection of concrete as a support medium should be the last resort. Nevertheless, where other support methods will not be effective concrete is an excellent solution.

The lining must be designed for these conditions:

- to support the external rock and water load that will be imposed during operation. In specific cases swelling rock may introduce loading.
- to ensure integrity of the liner under internal pressure. (conditions where seepage outflow is not of concern and concrete cracking without dislodging pieces is acceptable)
- to limit seepage outflow by limiting cracking to a tolerable limit.

Lining can be designed by utilising appropriate thick wall cylinder equation adjusted for the recognition of radial cracking in concrete and varying moduli values for rock layers behind the lining. Alternately, finite element methods are very suitable for this problem.

The thickness of the lining required depends on the size of tunnel and the hydraulic forces that will be applied. Generally thickness less than 0.2 m are difficult to place and thickness greater than 0.7 meter are rare. For majority of cases where concrete is required for stability due to external loading,reinforcement is not necessary. Unusual cases such as squeezing rocks, or very high external water pressure may however, require reinforcing. Reinforcement is not required for the internal pressure conditions unless severe cracking could occur resulting in dislodging concrete pieces specially in a dewatering conditions. However, if reinforcement is considered necessary, it is prudent to design to limit cracks to 1 mm. Thus deformations can be allowed that would result in a few concentrated cracks of up to 3 mm before reinforcement would be necessary. If reinforcement is considered necessary, them it should be designed to limit cracking to 1mm in width, with cracks distributed around the liner. Appropriate design methods are available to calculate the deformations and load sharing among the rock, concrete and reinforcement.

(h) STEEL LINED LENGTH

The length of the steel liner must be carried to a point which satisfies two conditions:-

- Hydraulic containment
- Acceptable hydraulic gradients

The requirement for hydraulic containment has been dealt with earlier. Once this is satisfied, then it is necessary to ensure that the hydraulic gradient from the end of the liner to the nearest exit point is low enough to prevent instability of soil or rock at the point of exit. The possible ways of developing such instability are:-

- uplift beneath impervious, soil or rock layers over lying more pervious rock
- erosion and piping of soil at the ground surface or at pervious layers within the soil.
- erosion and piping of a fault or shear zones existing at the ground surface or at an underground opening.
- Hydraulic jacking of rock blocks in to underground openings.

To eliminate these possibilities it is necessary to prevent the development of excessive water pressures by installing suitable drainage. No damage can occur provided that the high hydraulic pressures are dissipated in the rock marss near the end of the steel line. However it is essential to understand the geological condition and the relative permeability of the various materials in designing the drainage system.

(i) GROUTING

Grouting is undertaken over various portion of the tunnel for the following reasons:-

- Contact grouting. to fill large voids between a placed concrete lining or concrete back packed steel lining, and the surrounding rock. Low pressures are normally used
- Skin grouting. To fill the annular gap between a steel lining and its concrete encasement which forms due to concrete shrinkage, due to plastic set in the rock during loading and unloading, due to the differential temperature between the liner and the rock.

  Consolidation grouting to consolidate blast damaged or relaxed rock and to reduce leakage. Improvement of the modulus of rock is claimed by some investigators, but it is a debatable point.(see Fig.7.)

In a tunnel, grouting process commenced with contact grouting which is confined to arch area. Stable mixes 1:1 cement/water by volume or thicker with 0.5% to 1% bentonite is pumped using pressures of 3 kg/cm2 through holes spaced at 3m interval.

Skin grouting is normally the second step. Grout mixes must be thinner with about 2% bentonite to improve fluidity and penetration. Grouting should be done in rings (6-8 holes per ring) moving up slope with forward holes open to allow drainage. Pressures of up to 50% of the steel liner buckling pressures are appropriate to ensure against localised loading and buckling.

Consolidation grouting is the last step and is done by the ring method, moving upslope and grouting through the same holes as the skin grouting with the packer attached to the steel liner. A second grouting of the gap as well as the fractured rock can be achieved. Care should be taken to limit the maximum pressure to 75% of the buckling pressure. By this method in addition to consolidating the rock, it lends to induce compressive stresses in the concrete/steel liners.

Consolidation and contact grouting of a tunnel lined only in concrete is done to the same general criterion as a steel/concrete section. Leakage control grouting may not be effective if the leakage occurred as a result of hydraulic jacking.

5.0 OPERATIONAL ASPECTS

External Pressure Relief in Lining Systems is achieved by providing weep holes spaced at 3m unless special geological features dictates closer spacing. Flat concrete invert slabs are susceptible to uplift and even when used in unlined tunnels will require weep holes. Pressure relief valves in lining and header pipe systems can also be used, but clogging due to calcification and debris must be considered.

Plugs are typically located in construction access tunnels at the inter section with unlined and concrete lined pressurised waterways to control leakage. They must be designed to resists side shear and prevent excessive seepage around the plug. They are typically constructed with mass concrete and both contact and consolidation grouting are required.

Traps for collection of soil and rock are necessary for unlined or shotcrete lined tunnels, and where debris can enter from the intake or surge shaft. They should be located up stream of the concrete or steel lined portions. To reduce their first filling it is advisable to pressure wash the tunnel walls.

Invert treatment of unlined water ways ranges from simply removing excess tunnel muck to grading remanent muck and covering it with asphalt or concrete pavement. The advantage of removing or covering the muck is the reduction in material which can be transported along the tunnel during operation and therefore a reduction in the required debris trap volume. The advantages of a paved invert are reduced hydraulic roughness and headloss, a sound surface is provided for construction, inspection, and maintenance vehicles and increased allowable velocities.

Initial filling of the pressure tunnel should be done in a controlled manner. Filling the tunnel slowly allows pressure equalization to occur and thereby limit the deformation of the rock and liners. By careful monitoring of water levels in the surge chamber it may be possible to asses the leakage out of the tunnel or if leakage accelerates at a specific internal pressure hydraulic jacking or piping may be occurring. The surrounding area also should be monitored before and during initial filling. Dewatering of tunnel should also be done carefully as in the case of watering of the tunnel with necessary parameters monitored.

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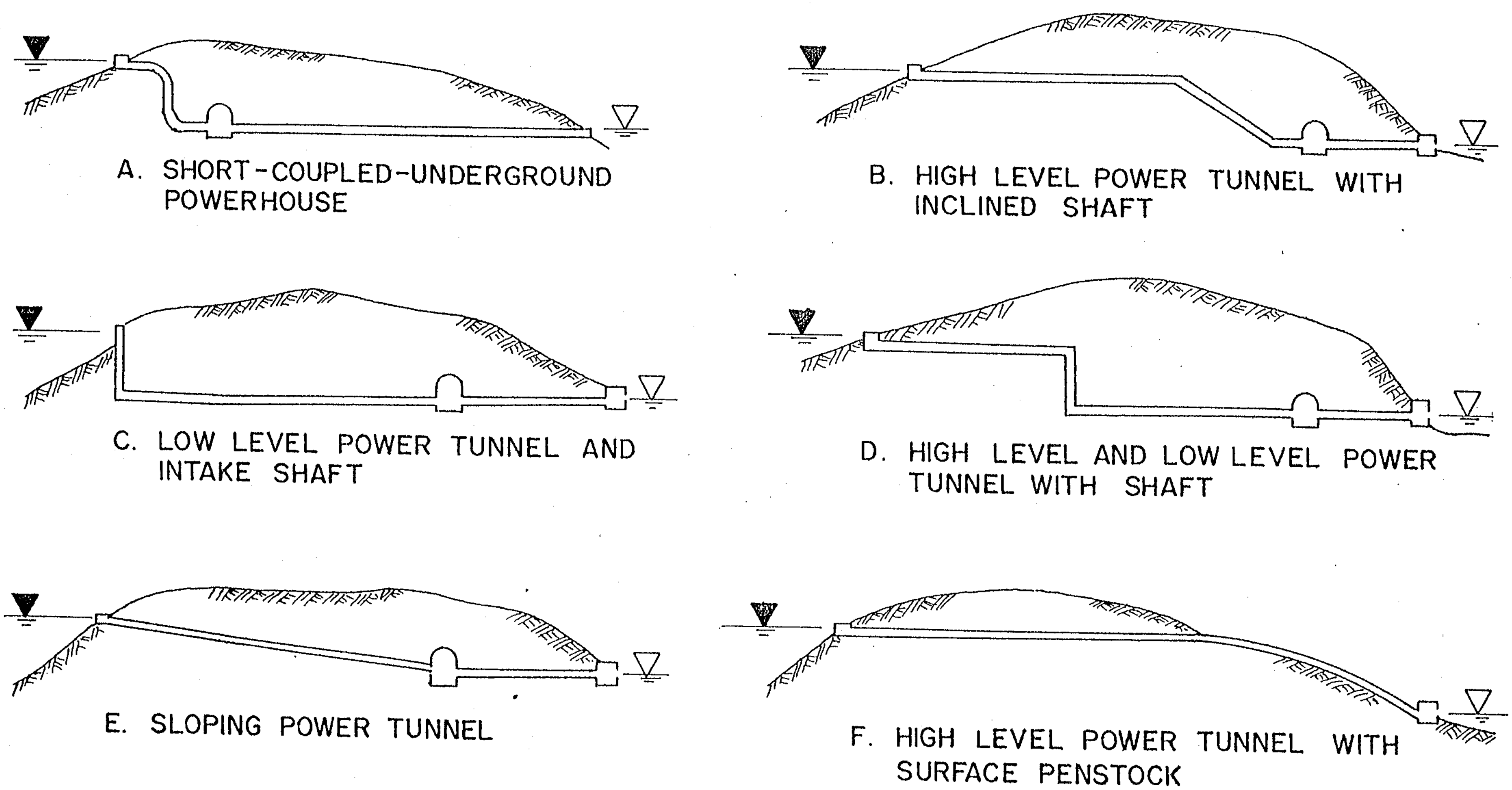
TABLE 1

GEOTECHNICAL DATA .

Geotechnical Data	Site Investigation
Preliminary information	Published and unpublished geological maps, reports, memoirs etc. Records, logs of excavations, boreholes etc. in close proximity to proposed project. Stereo-aerial photograph
Geological description	Surface mapping, core logging etc.
Rock material strength and deformation characteristics.	Laboratory testing (uniaxial and triaxial)
Rock mass index properties.	Assessment or rock quality (eg R.Q.D) Point load testing etc.
Discontinuity data : - description - orientation - spacing - continuity	Surface mapping Structural logging of boreholes Core orientation Mapping of surface outcrops and completed excavations
Discontinuity shear strength characteristics	In situ shear testing Shear box tests
Rock mass deformation moduli	Geophysical methods Borehole modulus gauges Plate bearing tests Monitored excavations
In situ stress measurements	Borehole stress meters Deformation gauges Strain cells
Unit weight	Laboratory testing
Groundwater conditions	Piezometric measurements Packer tests Visual inspection of excavation
Seismic data	Published seismic records Vibrograph measurements



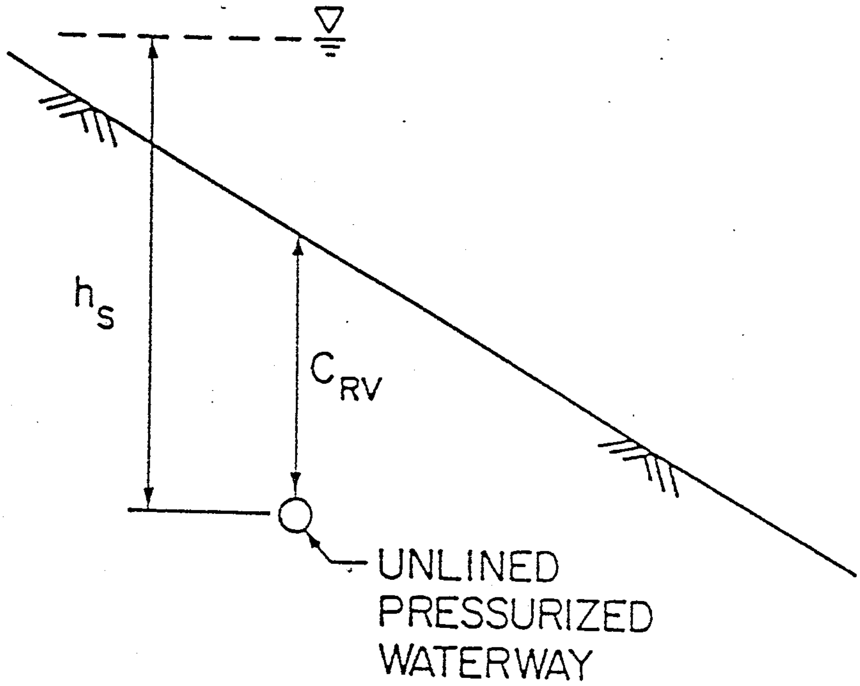
NOTES: SURGE FACILITIES NOT SHOWN

SINGLE TUNNEL ONLY - DISTRIBUTION SYSTEMS NOT SHOWN

SURFACE OR UNDERGROUND POWERHOUSE AS SHOWN

VARIOUS POWER TUNNEL SCHEMES

FIGURE - I



C_{RV} = x % OF h_s

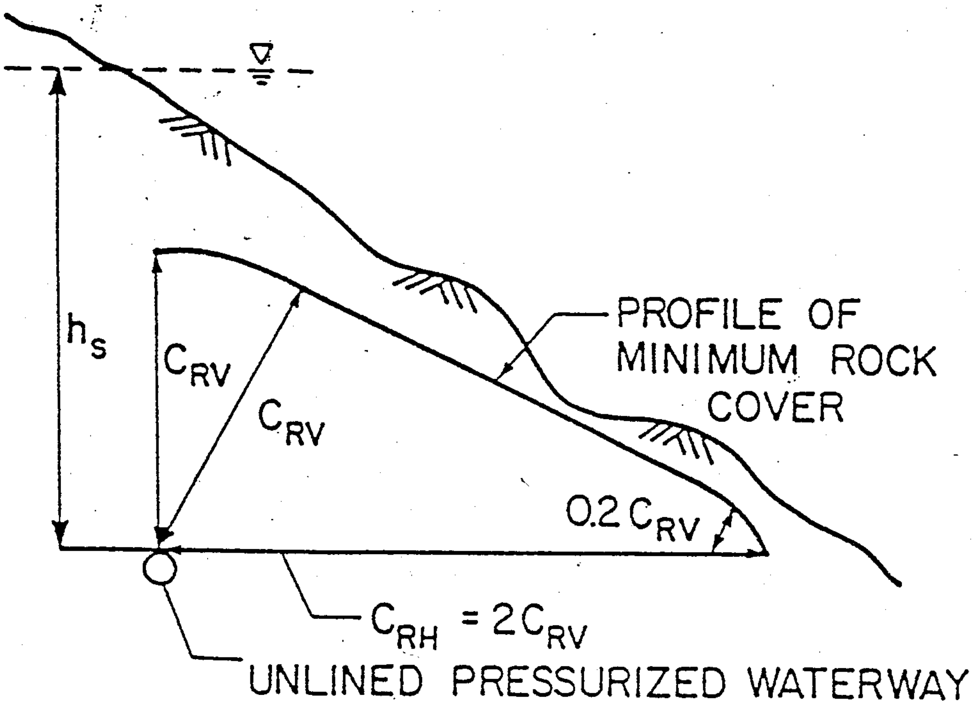
C_{RV} = VERTICAL ROCK COVER

x = 30 TO 100 %

h_s = STATIC HEAD

Vertical Criterion for Confinement

FIGURE – 2



$$C_{RV} = \frac{h_s \gamma_w}{\gamma_R}$$

C_{RV} = VERTICAL ROCK COVER

C_{RH} = HORIZONTAL ROCK COVER

h_s = STATIC HEAD

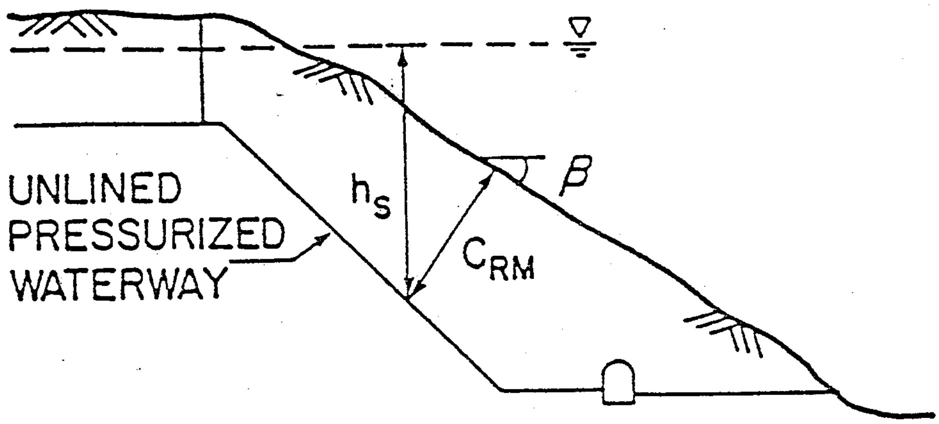
γ_w = UNIT WEIGHT OF WATER

γ_R = UNIT WEIGHT OF ROCK

Snowy Mountains Criterion for Confinement

Source: Adapted from Dann et al. [1964 (10)].

FIGURE-3



$$C_{RM} = \frac{h_s \, \gamma_W \, F}{\gamma_R \cos \beta}$$

C_{RM} = MINIMUM ROCK COVER

h_s = STATIC HEAD

γ_W = UNIT WEIGHT OF WATER

γ_R = UNIT WEIGHT OF ROCK

β = SLOPE ANGLE (VARIES ALONG SLOPE)

F = SAFETY FACTOR

Norwegian Criterion for Confinement

Source: Adapted from Bergh-Christensen and Dannevig [1971 (11)].

FIGURE-4

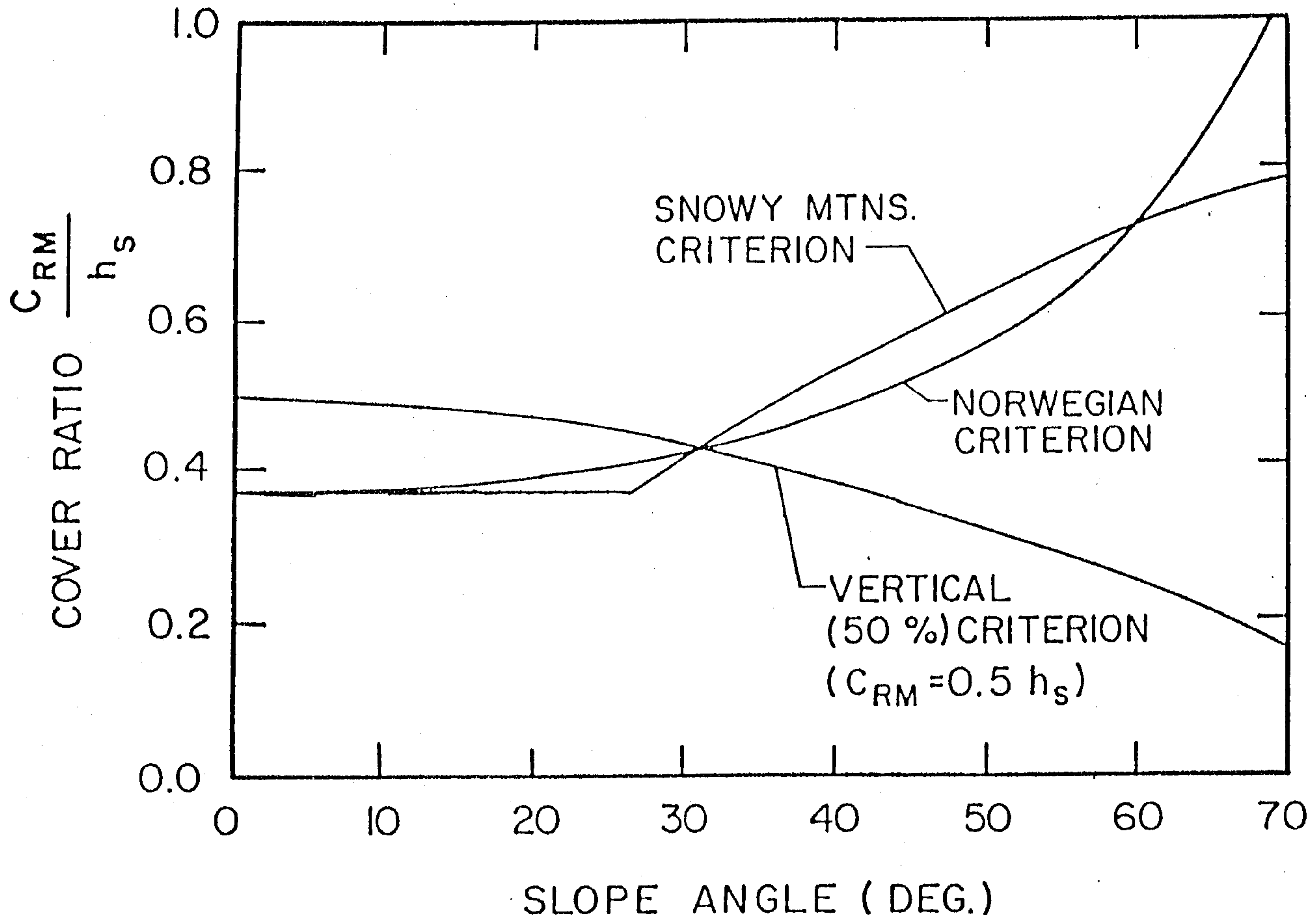
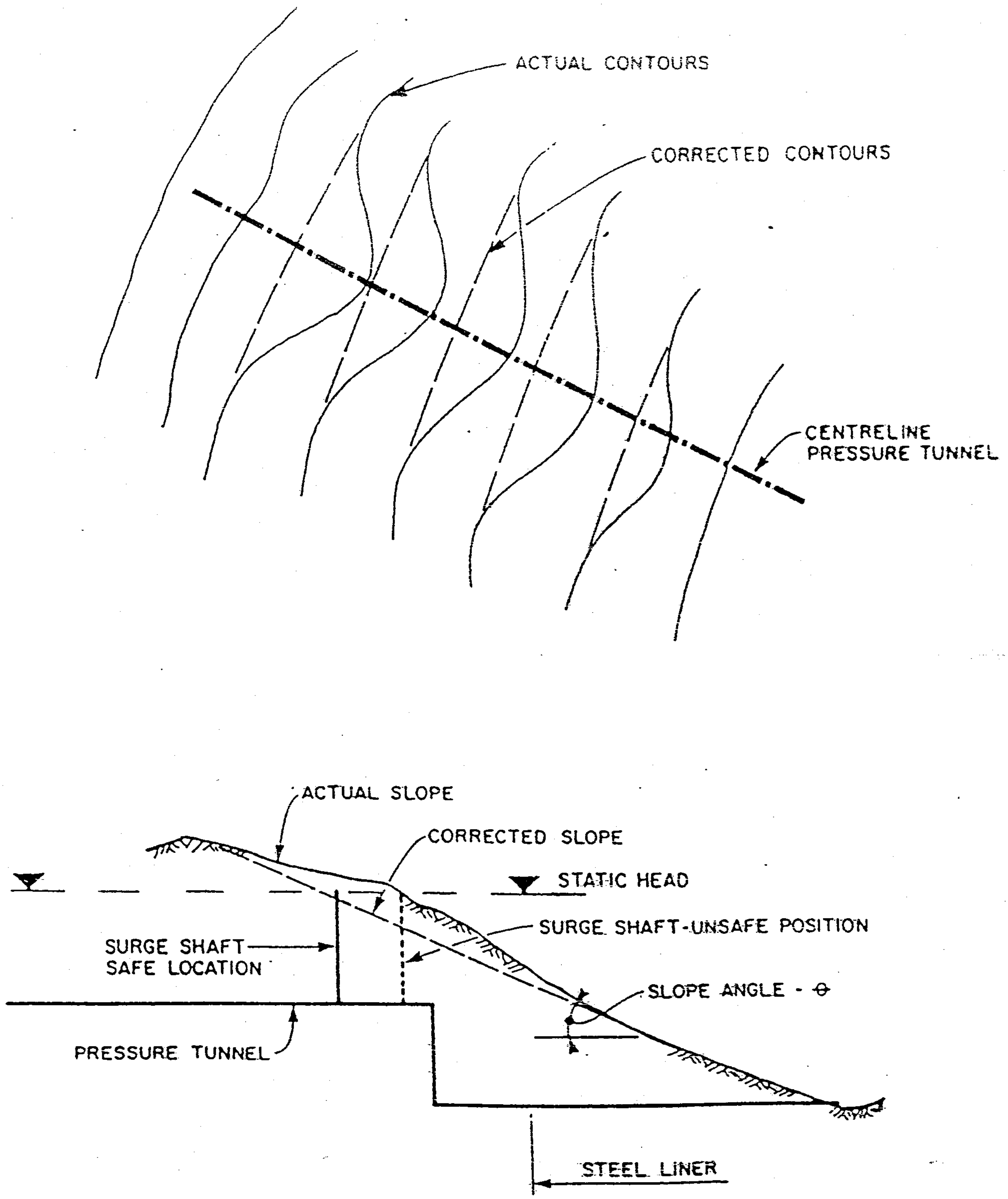


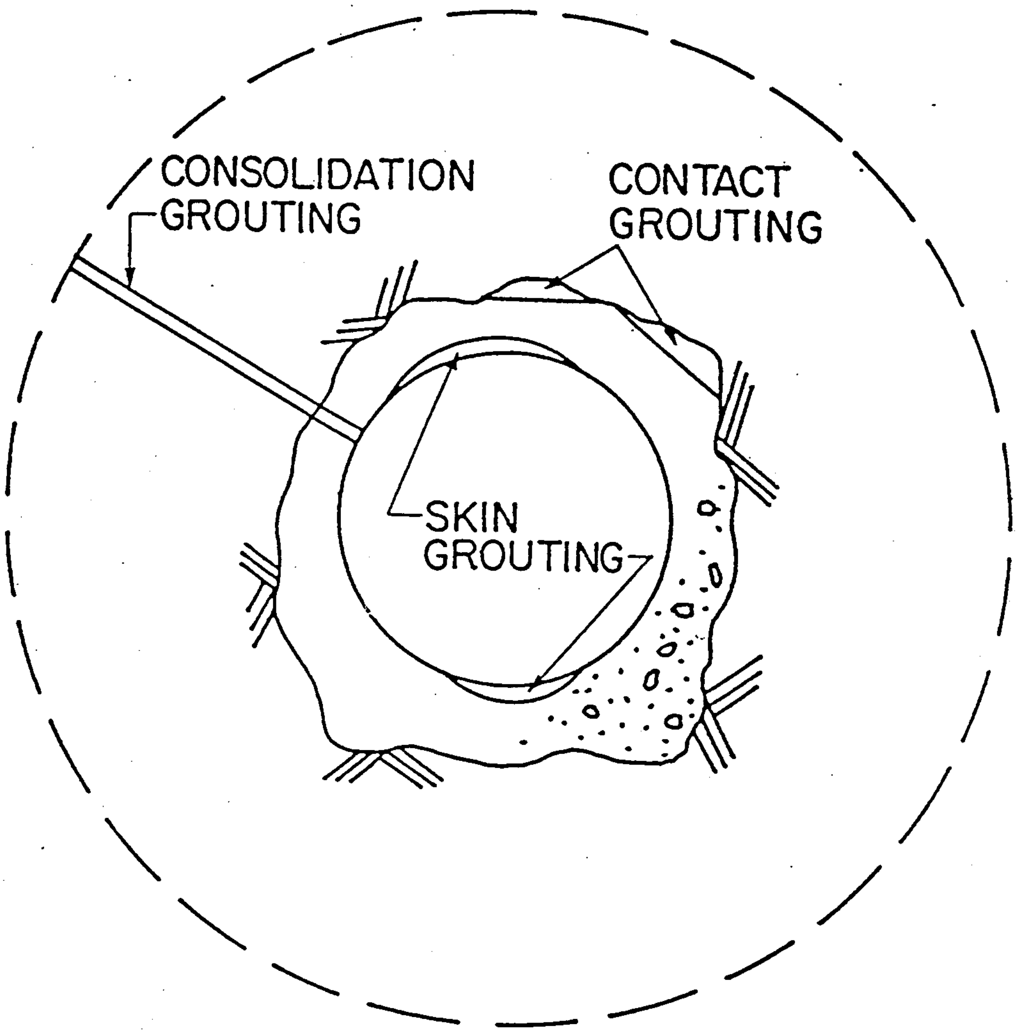
Figure 3-8. Comparison of vertical, Snowy Mountains, and Norwegian criteria for sloping topography.

FIGURE- 5



BROCH CORRECTION FOR TOPOGRAPHY

FIGURE – 6



Skin, Contact and Consolidation Grouting

FIGURE-7

APPENDIX I

HEADLOSSES IN TUNNELS.

Frictional head loss in waterways due to surface roughness are typically determined using the theoratical Darcy - Weishback equation. (see below). Other head losses include intake. trushracks. elbows. expansions. contractions and mainfold. These can be evaluated as described in design of small Dams - USBR. Fig 8 is a pictorial representation of these losses. Fib 9 indicates methods which can be adapted to recude head losses in pressure tunnels.

Darcy-Weisbach: $$h_L = \frac{f\,L\,V_T^2}{2\,D\cdot g} \qquad (6\text{-}1)$$

Manning: (metric) $$h_L = \frac{L\,V_T^2\,n^2}{R_h^{1.33}} \qquad (6\text{-}2)$$

where:

h_L = head loss

L = length of waterway

V_T = flow velocity

R_h = hydraulic radius of waterway

D = diameter of waterway

g = gravitational acceleration

n = Manning roughness coefficient

f = Darcy-Weisbach roughness coefficient

(a) Headloss Formulas

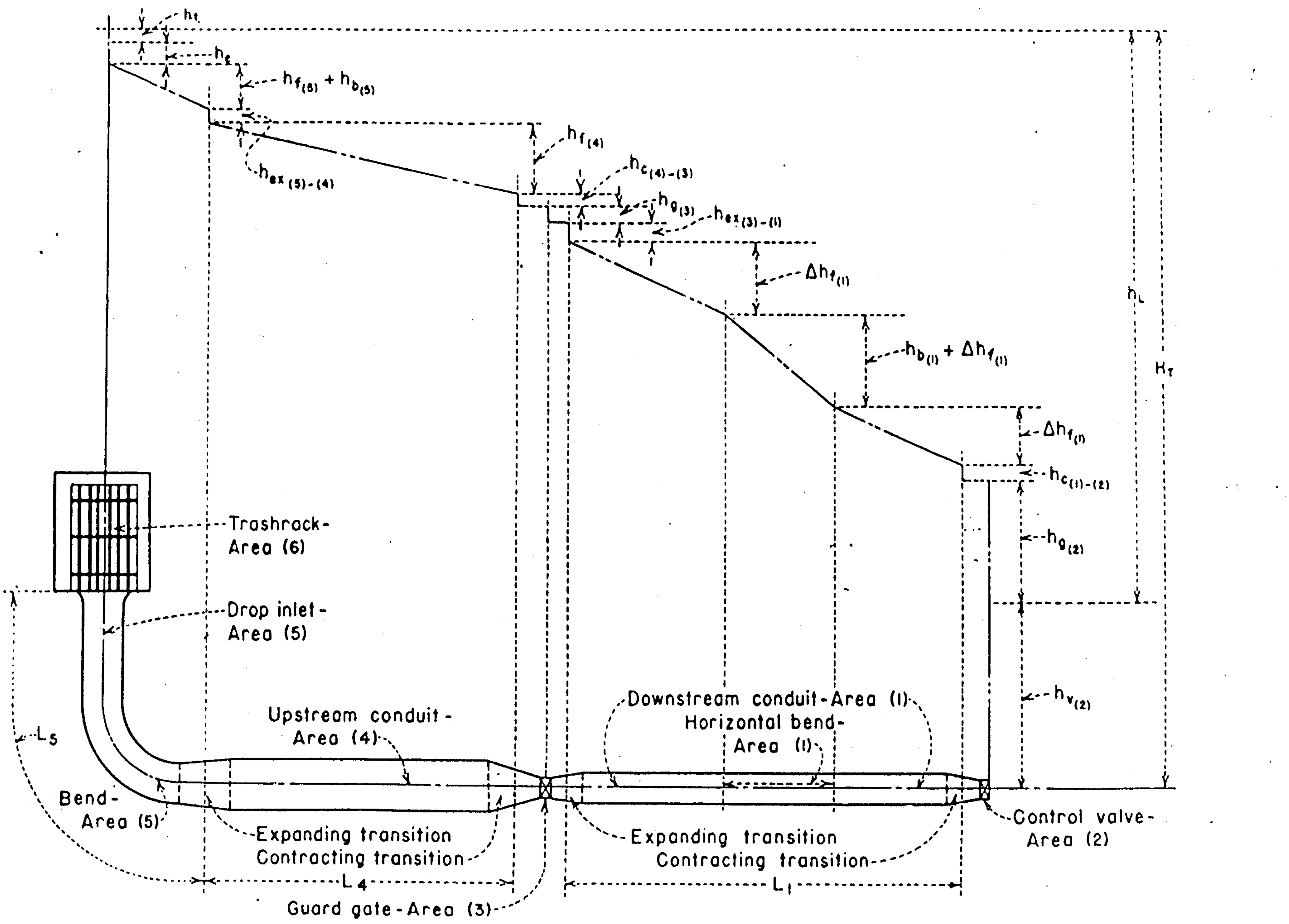
Darcy-Weisbach: $$f = [2\log(14.88\,\frac{R_h}{K})]^{-2}$$

Manning: (metric) $$n = \frac{K^{0.667}}{25.4}$$

where: K = measured roughness (in meters)

(b) Roughness Coefficients

Formulas for frictional headlosses and roughness coefficients due to surface roughness.

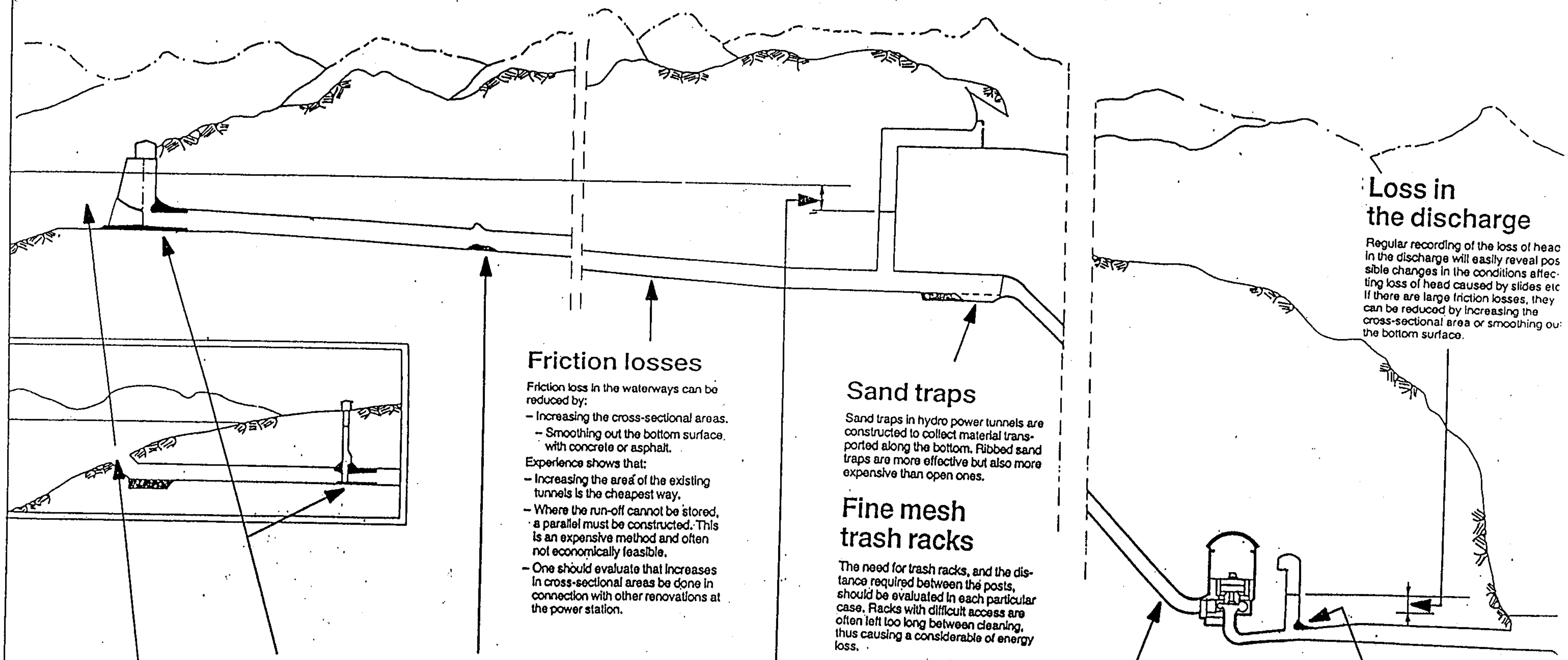


Pictorial representation of head losses in conduit flowing under pressure.

SOURCE - Design of small dams
USBR

FIGURE - 8

Reduction of energy loss in water-Ways at high pressure hydro power plants



Loss in the discharge

Regular recording of the loss of heac in the discharge will easily reveal pos sible changes in the conditions affec-ting loss of head caused by slides etc If there are large friction losses, they can be reduced by increasing the cross-sectional area or smoothing ou: the bottom surface.

Friction losses

Friction loss in the waterways can be reduced by:
- Increasing the cross-sectional areas.
  - Smoothing out the bottom surface. with concrete or asphalt.

Experience shows that:
- Increasing the area of the existing tunnels is the cheapest way.
- Where the run-off cannot be stored, a parallel must be constructed. This is an expensive method and often not economically feasible.
- One should evaluate that increases in cross-sectional areas be done in connection with other renovations at the power station.

Sand traps

Sand traps in hydro power tunnels are constructed to collect material trans-ported along the bottom. Ribbed sand traps are more effective but also more expensive than open ones.

Fine mesh trash racks

The need for trash racks, and the dis-tance required between the posts, should be evaluated in each particular case. Racks with difficult access are often left too long between cleaning, thus causing a considerable of energy loss.

Intake

The costs of loss of water increase with the height. Therefore the water level above the inlet should be kept as high as possible without losing water. Intakes that can entrain air should be avoided since air in the head race tun-nel can cause increased loss of head, unstable conditions and harmful blow outs.
To avoid unnecessary losses, the trash racks should be checked and cleaned regularly.

Gate plugs and concrete linings

A good hydraulic design will reduce losses considerably. Use should be made of:
- Diffuser angles between 6° and 10°.
  - A truncated diffuser rather than lar-ger diffuser angles.
- Defined separation point for the jet.
  - The end area normal to the tunnel wall.

Slides

Slides inside a tunnel will cause reduc-tion in cross-section and thereby loss of head. With per-manently installed instrumentation it will be possible to determine whether there has been a slide and the size of the loss created.

Repair work at a suitable time is an economic question.

Loss in intakes and head race tunnels

The difference in water levels be-tween the resevoir and the surge chamber is measured with equipment such as floats. Changes in the loss conditions can mean blocked trash racks or slides in the tunnel.

Penstock

Generally there is little that can be done to reduce losses in the penstock other than normal inspection and maintenance. Nevertheless, these los-ses should be measured and an eva-luation made of any possibilities of improvement.

Draft tube outlet

The sudden increase of the cross-sectional area at the outlet of the draft tube creates losses. If the design of the transition area is hydraulically cor-rect (as shown in the figure), these losses will be reduced.

FIGURE - 9

NORWEGIAN HYDRODYNAMIC LABORATORIES

RESERVOIR WATER-TIGHTNESS

Eng. R.L.De S.Munasinghe

Reservoirs have been and will continue to be constructed in various types and sizes and for a variety of purposes. They are essentially constructed to store water abundantly available during certain periods of time and for usage during lean or dry periods. If this is the purpose of a reservoir, then it is imperative that the reservoir should be water-tight. Water-tightness may however be within tolerable limits depending on the value of the water that might be lost by leakage. This value depends on the purpose for which the water will be stored and also the availability of the water. Tolerable limits will also depend on the problems that may be created by the leakage.

To determine whether the leakage from a reservoir to be constructed would be within tolerable limits, one has to determine the permeability of the rock strata forming the impounding basin. Seepages through the sub-strata will among others (a) cause loss of water, (b) initiate the process of erosion which in the course of time could endanger the stability of the dam and any weak locations of the reservoir perimeter and (c) produce uplift pressures on the structure particularly of the dam. Determination of rock permeability therefore should form a major part of the fundamental investigation in reservoir projects.

Any of these three effects of seepage or a combination of two or more of them could possibly turn out to be a major decisive factor on the feasibility of a project. Seepage beyond limits, could cause the project to be uneconomical, since cost of excessive sealing works are generally very high. A reservoir with uncontrolled and intolerable seepage through the dam foundations or as a matter of fact, through any part of the reservoir rim, can in no way be considered an efficient project. If due to this seepage, the reservoir cannot be filled to capacity, the project is unsuccessful; if the purpose of the reservoir cannot be fully achieved due to these conditions, then the project is inefficient; and until the reservoir is treated to tolerable limits of seepage, the project is incomplete.

An investigation programme with emphasis towards water tightness of a reservoir would bsically center around the geology and would cover

1. Stratigraphy and Lithology
2. Tectonics
3. Weathering Pattern
4. Hydrogeological Conditions and
5. Other Physical and Geological Features.

Study of stratigraphy and lithology, tectonics and also the weathering pattern can be conducted by surface geological mapping and bore-hole drilling and the study of aerial photographs. The effects of these on the water tightness of a reservoir can be determined only by Water Pressure Tests (WPT) conducted during drilling operations. Hydrogeological conditions can be ascertained by rock permeabilities determined through WPT, but for a clearer and a more accurate understanding of the situation, a study with the help of piezometers are necessary.

Project investigations should be performed well in time and well ahead of the construction programme and long before the contractual procedures are set in motion. No amount of excuses would suffice for entrusting contractors to conduct investigations for purposes of designing cut-off works, since investigation data form the core of the deciding factor on the scope of the project. They could be the basis on which a final decision on whether the project should be implemented or not. Observational approach which depends on hind-sight is not believed to be the right approach. Timely investigations and an approach with fore-sight, as would an engineer approach a problem, is considered appropriate.

Quoting from "Rock Grouting with Emphasis on Dam Sites" by Prof. Karl Ewert (1985),

"It seems advisable to outline the most important aspects for investigation programmes related to permeability of the sub-strata as well as the for the practical execution of the grouting works and subsequent control measures.

An investiagtion programme should begin with a study of the following complexities:

The basic question to be studied will be whether there is another valley in the neighbourhood of the reservoir area. It could drain off water from the reservoir if there were any permeable rock zones in between functioning as a hydraulic connection if the original water table were below the intended reservoir level.

This complexity cannot always be cleared by geological mapping alone, which is particularly true of the position of the groundwater table. Whenever the groundwater table seems to fluctuate below the reservoir level, piezometers should be installed. It is also within the scope of hydrogeology to find out whether the groundwater table beneath the slopes adjacent to the dam site rises steeply or proceeds into the abutments on the level of the river. A high or deep position of the groundwater table gives valuable information about the average permeability of the rock mass and helps to define the adequate extensions of the grout curtain eventually needed.

The formation of water carrying openings along joints and other types of dicontinuities is a long-lasting process. Once a certain network of paths has developed, the direction of groundwater flow cannot change easily, and it still maintains its influence even under the conditions of an impounded reservoir.

This calls for the early installation of a sufficient number of piezometers already in the phase of preliminary investigations. It is highly advisable to measure the groundwater table and its precipitation-depedent fluctuations over a long period including at least one dry and one rainy season. It is also important to provide for a good graphical representation of the readings, maps of groundwater contours and piezometer hydrographs should be plotted. The interpretation of maps and hydrographs permits conclusions concerning

- Average permeability of the rock
- Extension of an eventual grout curtain
- Sections which can possibly be excepted from the treatment
- Existence of only one or more groundwater regimes also including sections with perched water.
- Natural reaction of the groundwater to the precipitation (particularly to interpret groundwater behaviour during impoundment)"

The ground water conditions at a project site (dam site, reservoir or its perimeter) could be objectively studied by the use of a network of piezometers. Such a system would reveal the behaviour of the ground water table in response to the adjacent water bodies - may be the river - and the change in climatic conditions. This is a very important aspect to be studied particularly in weak locations on the reservoir perimeter. Any indications of a fluctuating ground water table or a flat ground water table should not go unheeded. Special attention to such indications are called for especially when they come in the form of recommendations or warnings.

Underground permeability values are generally based on the results of Water Pressure Tests. The method adopted in conducting a WPT is the injection of water under pressure into a borehole stage by stage. The stage examined is sealed off from the balance of the bore-hole. WPT in its general form is conducted with increasing and decreasing pressure steps, eg. 2 - 4 - 6 - 4 - 2 bar. Selection of pressures on a WPT need to be selected with care since excessive pressures could irreversibly increase the perviousness of the rock by hydraulic fracturing. Hydraulic fracturing could reduce or cause loss of impervious qualities of a rock mass. However, it is important to clearly understand the pressure dependent behaviour of the permeability, particularly in the case of high dams which would impound great depths of water. Impounding such depths of water could during the course of impounding, cause hydraulic fracturing in areas (may be under the dam and also on the perimeter of the reservoir) of weakness in the sub-strata. The pressures at which WP Tests are conducted are varied during the injection of the water in order to study the permeability behaviour which may be dependent on pressure. The determination of the dependency of permeability on pressure is considered important particularly in the case of reservoirs constructed to impound deep water.

Permeability testing programmes could be adopted to assess the water conductivity as well, within the rock mass with the impounding of the reservoir. This assessment would facilitate the decision on the type and manner and even the magnitude of the sealing works if the need for such works prevail. Such sealing works may be required in other locations on the reservoir perimeter, not only in the dam foundations.

Loss of water from a reservoir could be estimated with sufficient or generally acceptable accuracy on an average value of the coefficient of permeability. But there is the more serious aspect of erosion of the rock mass through which water - due to impounding of the reservoir- will flow under a much higher head of water than that the rock mass had ever experienced before. This requires a more accurate measurement of the water conductivity since erosion is dependent on the volume of flow and the velocity of flow of water through the available water paths and also on the erodibility of the rock upon its mechanical and chemical resistance.

Water conductivity of a rock mass is a quality that is complicated and difficult to measure with accuracy due to the heterogeneity of the rock specifically in relation to the paths available through it for the passage of water. Nevertheless the importance of ascertaining the permeability as accurately as possible and as far as possible the prevailing pattern of water conductivity, is not in doubt, since it gives a measure or at least an indication of the degree of water loss, and other adverse effects due to the passage of water.

In dam and reservoir construction important decisions are based on the results of the water pressure tests. With these tests, decisions are taken on sealing or cut-off works which greatly contribute to the economic significance of the project. Technical feasibility of a project is based on other factors such as the hydrology and the topography etc. etc.. It should therefore be ensured and established that the testing techniques and the evaluation of test data give information of reliability.

To assess or estimate the seepage losses and velocities of percolating water, a unit of measurement or a characteristic coefficient need to be established. The Coefficient of Permeability (k) indicating the velocity (cm per sec.) of water through a porous medium is calculated from the water takes at Water Pressure Tests. The conversion of WPT results into this coefficient is difficult. Therefore, an independent or an absolute absorption value is used as the characteristic value is necessary. It has become the general practice to regard a critical rate of infiltration of water per metre section of the bore-hole in one minute as a characteristic value. It is common practice now to use the Lugeon Unit for the critical absorption rate. One Lugeon Unit corresponds to Q(wpt) = 1 Litre per metre per minute at 10 kg per Sq. cm. (10 bar). One Lugeon Unit is approximately equivalent to k = 10 cm/Sec. Determination of the characteristic values of permeability coefficient or the independent or absolute absorption rates or Lugeon Units are attempts to establish a scale by which the rock mass permeability could be measured. This would facilitate assessment of the sealing measures required to be imposed.

A river is an open path of water along which water flows with no restriction. The construction of a dam across the river causes this path to be sealed. With the closure of its main open path, the water begins to head up causing impounding of deep waters as planned by the engineers. With the water impounded to large depths, the pore pressure in the perimeter of the reservoir begins to rise subjecting the banks to conditions that it had not been subject to before.

Under normal conditions one would expect the groundwater table to closely follow the topography of the land. If it does not, then the groundwater regime should necessarily be studied and studied very carefully. Springs of water issuing from hill slopes give a very good indication of the elevations at which the ground water table exists. The actual elevation at which springs appear should be taken note of. There have been locations where springs have appeared at hill tops while the lower elevations of the hill slope were totally dry. such a situation should prompt an engineer of the possible existence of a perched water table and not the normal ground water table.

Preliminary investigations and designs, hydrological investigations during long periods of time, detailed investigations and tender designs are those that are usually conducted prior to the commencement of construction activities. These phases of investigations and design are conducted through very long periods of time. It may sometimes be 10, 20, or even 30 years depending on the magnitude and the complexity of the project. During this period many experienced engineers would be involved with the preliminary investigations and analysis of data etc.. The conclusions arrived at and the rcommendations made by them should necessarily be given their due recognition, particularly when warnings of prevailing adverse conditions have been made.

The general understanding with regard to the weathering pattern of a rock mass is that the degree of weathering decreases with depth since the agencies of weathering are air and water from the ground surface. There could however be other agencies of weathering such as hot fluids emerging from within the earth. The effects of these hot fluids could be dissolution of certain minerals and rocks and thereby the formation of cavities and alteration of certain other minerals. These effects increase with depth unlike in the case of weathering by air and water. All these effects cause improved water conductivity within the rock mass. Therefore the water permeability and hydro- geological conditions should be a matter of prime concern as the success of the dam and reservoir would depend much on the water tightness.

Tectonic faults and jointing, karstification and solution cavities, surface weathering and hydrothermal cavitation and alteration are those that could cause permeability and hydraulic conductivity in a rock mass. When rock masses that have suffered due to one or more of these effects form the reservoir impounding basin, loss of water by leakage could and should be expected. For making optimum use of the stored water, these leaks should be sealed and the dimensions and the type of the sealing works should be pre-determined with adequate and timely investigations. The equipment required to implement the designed sealing works should be made available or even manufactured to suit the works designed. The sealing works should never be curtailed to match the available equipment.

Sealing works are usually provided under the dam foundations and the sealing is normally achieved by means of a grout curtain. There are of course other methods of sealing leakages. But to construct a grout curtain or other sealing works, one should be aware of from where and to where the water would leak. If sealing is to be effected by a grout curtain, then it is a fundamental requirement that the two ends of the grout curtain should be tied to impermeable strata for it to be effective. It should also be anchored well into impermeable strata along its bottom. The layout of a grout curtain and the media to which it would be tied should be well defined after adequate investigations. This is fundamental and common sense. Searching for a media to tie up a grout curtain at its completion stages is not considered good engineering practice. A hanging grout curtain in an area whose permeability or the water coductivity qualities are not known will not serve its purpose.

Engineers and geologists together would design the layout of a drilling investigations programme required to gather specific information on the underground conditions in the project area. This layout may be based on geological surface mapping which by itself would not reveal the underground conditions. The information required are for specific purposes. Jamming of a drill string or the difficult conditions at drilling sites should not be made an excuse for terminating any part of this investigation programme. Repeated jamming of the drill string in any bore hole should indicate adverse underground conditions. Such occurrences in adjacent bore holes should prompt the engineer of serious conditions underground and it is prudent for the engineer to go into further details to more accurately establish these conditions. Termination of the programme of investigation should not be the solution to a problem of this nature. Further, if for any reason of urgency, a part of an investigation programme had been transferred to another section of the proect, lack of time or funds or both should not be an excuse for terminating that part of the programme.

Quoting once again from Prof. Karl Ewert's "Rock Grouting with emphasis on Dam Sites".
"A discussion of the complexity of rock permeability for geological purposes in dam construction must deal with the question of application of the absolute absorption rates have, ie. if they are suitable for serving the tasks defined at the outset, which are to determine the seepage losses, the erosional behaviour, the reduction of uplift pressure and the grout takes. Before this is done, the basic question should first be dealt with; namely, what amount of seepage loss can be tolerated, and for which reasons, or to what extent and why they must be limited.

Several authors have been occupied with the permissible rock permeability for dams and have attempted to set limiting rates on the basis of water pressure tests (Table 15) - Annexed.

Regardless of the widely differing views of the permissible permeabilities, the reasons are not made completely clear in the literature. .....

Houlsby (1976) suggested a more differential application of the Lugeon criterion, considering not only the type of dam but also the relationship between the value of the water and the expenditure for impermeabilization (Fig 99)" - Annexed.

Table 15. Impermeabilization criteria based on WPT rates proposed by various authors

Author	Tolerable WPT rate proposed		Conversion to a common pressure basis	
	WPT rate ($l\,m^{-1}\,min^{-1}$)	Pressure (bar)	WPT rate ($l\,m^{-1}\,min^{-1}$)	Pressure (bar)
1. Lugeon (1933)				
a) $H \geqslant 30$ m	1	10	0.3	3
b) $H < 30$ m	3	10	0.9	3
2. Jähde (1953)				
a) Grout holes	0.1	3	0.1	3
b) Control drillings	0.5–1.0	3	0.5–1.0	3
3. Terzaghi (1929)	0.05	0.1	1.5	3
4. Keil	0.2	3	0.2	3
5. Blatter	0.33	10	0.1	3
6. USA	3–4	10	0.9–1.2	3
7. USSR				
a) $H \geqslant 10$ m	0.05	0.1	1.5	3
b) $H < 30$ m	0.03	0.1	0.3	3

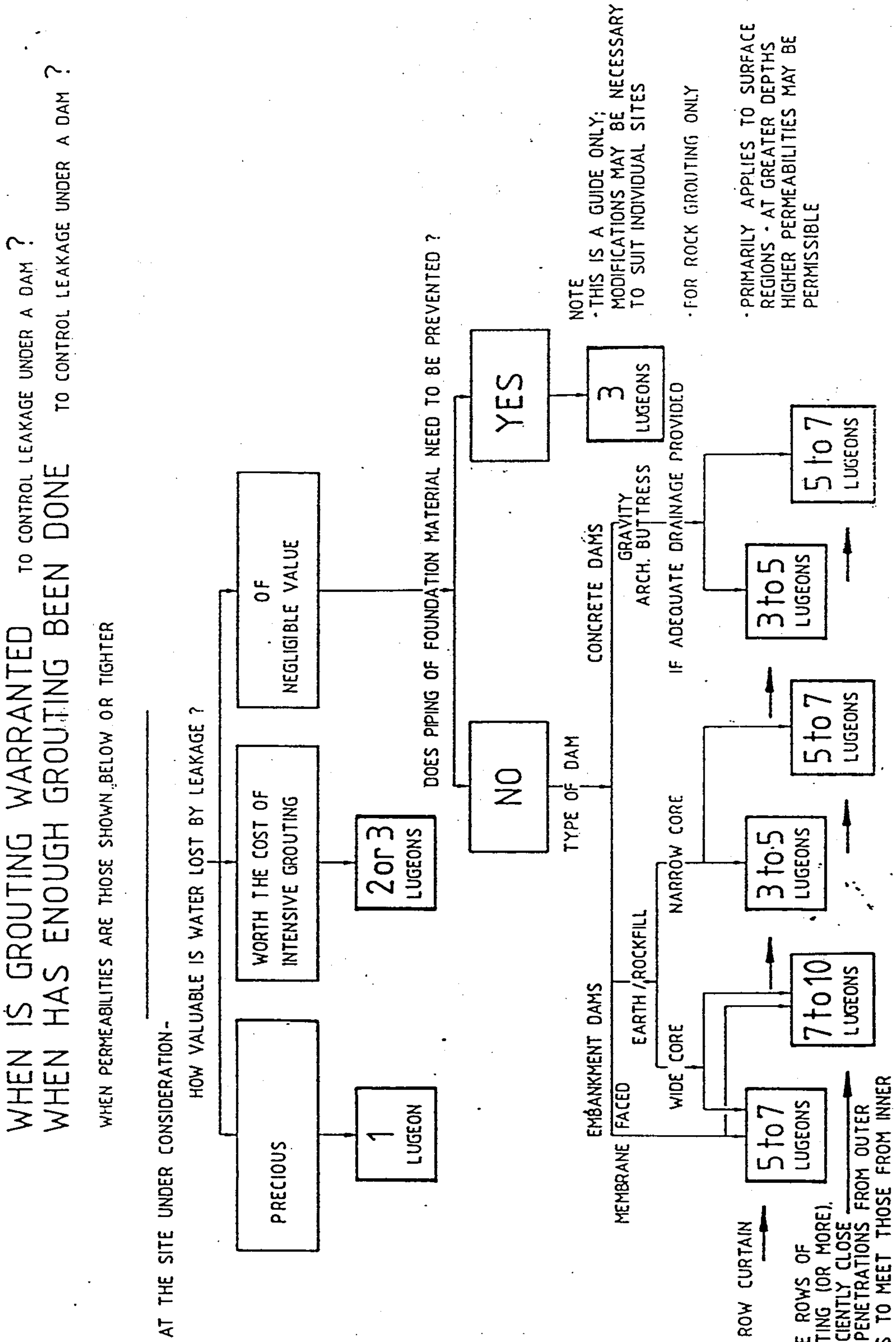


Fig. 99. Impermeabilization criteria on the basis of WPT rates proposed by Houlsby (1976)

Problems encountered in Dam Foundations & Remedies*

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The function of a dam is to transfer the water load to the foundation and ground below or to distribute the load partly to the sides by arch action. For a narrow valley of good rock an arch dam is best. For a wide valley with poor ground a 1:3 base to height ratio rock fill is best. The key to success is selection of the correct type of dam. The main criteria for a successful design are:

a. configuration of valley
b. projected water pressure
c. type of soil or rock
d. probability of seismic activity

Dams fail principally due to 2 causes

a. over topping by unexpected flood
b. foundation failure

If the subject is limited to the latter the failure of a dam can be due to several reasons.

Dams are thrown across rivers to stop the flow and store it for controlled use later. Rivers are found in valleys where rainfall run off collects to form identified surface drainage paths.

Streams and river beds etch the earths surface and either erode it further or cause siltation and build up depending on the slope of the land. Dams of necessity have to be founded on such terrain. If the terrain consists of alluvial deposits they have to be removed for good construction if the bed is rock the foundation is ideal for a concrete dam. But if the foundation is fragmented or decayed rock or karst then the problems like heavy seepage, uplift, solution or even crushing can occur.

The increasing height of dams can mean even the rock on which it is founded can be stressed beyond its limits and cause mechanical movement that will trigger new uplift pressures or lubricate a potential shear zone to initiate a fracture.

The Malpasset arch dam over river Reyran near Frejus suffered from such a scenario before it failed on 2.2.59. Foundation inadequacies also caused the failures of Vaiont dam in 1964 and Teton dam in 1979. On the positive side these failure also focussed attention to the development of new fields of technology as rock mechanics and engineering geology.

The skills thus developed have given the dam engineer greater confidence to combat more and more difficult foundations on which taller dams could be established.

A safe foundation implies that the dam engineer must obtain good foundation data. He must then interpret geological data in terms of physical and mechanical terms. From this basic premise it is clear that the following sequence is adopted for a safe structure.

(i) good foundation investigation
(ii) design of foundation to suit.
(iii) treat inadequacies in foundation to achieve some known or stable foundation condition.

The geologist's and hydrogeologist's services in this phase of investigation is never complete. Recent developments in engineering geology and rock mechanics have perfected drilling techniques, adits, bore hole samples, permeability and resistivity techniques to give a 3 dimensional picture of the geology of a dam foundation.

Dam axis geology is a "sine qua non" for the design of the dam. But the regional geology is equally important if the reservoir bed is to hold water. The feasibility of the reservoir can be in question if the regional geology is downgraded as happened at Samanala Wewa where numerous studies had indicated heavy seepage, yet the full water tightness of the bed had not been achieved before impounding commenced.

One could discern from the Samanala Wewa example the following weaknesses which may be peculiar to Sri Lanka.

(i) Period of investigation too long commencing in 1958. Long gestation periods, interests ebb and flow with political change, but is ideal for engineering investigation and planning.

(ii) Too many consultants from around the world. Some aspect is downgraded by one and prioritized by other .

(iii) Changing authority having control of the project in 1958 the Irrigation department to the C.E.B. in 1975. This leads to loss of historical data with physical movement.

(iv) Absence of continuity even in the said departments or boards by ad hoc changes, that result in loss of valuable data and information.

Other Examples of Peculiar foundation problems

Sampan Baru Dam - Indonesia

This was built by the Japanese in occupied Java but failed in 1945. The dam is 786 m long and meant for hydropower development.

Dam foundation is pumice which is a highly elastic and consists of a mix of airborne rock dust and lava. Pumice has a modulus of elasticity 60 times that of concrete. A dam built on such foundation rebounds and reverberates for each drawdown or any seismric activity.

Verney Dam - France

This is the lower dam in the Grand Maison pumped storage project in the French Alps. The power produced is 1800 MW. The Verney dam is 42 m high 430 m long and built on compressible aluvium foundation up to 80 m deep. As the foundation is permeable a diaphragm wall of clay and cement was installed to a depth of 47 m. The wall is capped by a R.CC. beam in order to keep the dam stable. In case of fracture of the wall the u/s foundation is composed of a 2 m thick impervious layer of moronic earth (0-125mm). To improve the imperviousness a reinforced bituminous geomembrane "Coletanche NTP 4" has been laid in the centre of the 2 metre layer. Dam was constructed with local aluviun and a bituminous concrete facing with 10 cm support layer and 2 water tight layers of 6 cm each.

Khao Laem Dam - Thailand

Rock fill Dam over Que Noi at Tambon started in 1972 is a 92 m high, 1,000 m long dam, built on a major fault zone. Storage is 8860 MCM.

The foundation is a diaphragm wall on calcareous limestone and a grout curtain. The R.B. is like a giant porous sponge and rivers used to appear and disappear underground like the Paragala ganga in Pimbura in Sri Lanka.

The water tightness was secured in such terrain by extensive grouting. There were 377,000 grout holes dug at 1 1/2 m spacing to average 15 m depth and an interlocking pattern adopted to fill cavities. In 1984 prices, project cost 9 Billion Baht. Civil works cost 6 B. Baht and of this 3 B. Baht went for foundation treatment which included constructing 6 galleries for seepage, total 21 km in length. There were 9 different fault zones in the damsite, most prominent being 3 pagodas fault running from Burma to Gulf of Thailand.

Lar Dam - Iran

1.3 km long, 105 m high had a chequered history of grouting. Investigation after completion of dam and grouting revealed fractured karstic caverns up to 800 m below dam crest. One single cavern measured 23 m high 67m wide and was at 300 m below dam. Dam is designed for a capacity 960 Mm^3. Construction was completed in 1980. Around 1/3 of the inflow of 40 m^3/sec is lost as seepage. Later investigation disclosed water tight formation is at 2000 m below the dam.

Zeuzier Dam - Switzerland

Good monitoring of dam behaviour led for the prompt action taken to save Zeuzier dam in Switzerland. This dam is 156 m in height and built on La Lienne a tributary of the Rhone river. The dam was built in 1957 for 82.8 mw power production. In 1978 one of the pendulums fixed for monitoring swayed after behaving regularly for 21 years. The reservoir was drawn down in 1979 and a team of geologists examined the dam after the snow had cleared. Investigations revealed, a tunnel for a road project had been driven 1.4 km away from the dam and 400 m below crest level. The tunnel excavation had caused zeuzier dam crest to move 110 mm. The tunnel had penetrated 3 km into vertically bedded jurassic limestone draining up to 10% the reservoir storage.

This led to the dam foundation rock to settle and the abutments to press on the concrete arch. When tunnelling was stopped the dam had settled 110mm and the gorge had moved 60mm inwards and a downstream rotation of 20mm. All these movements left numerous cracks on dam. Punching of the ground water reservoir was the cause of damage. Repairs were done by injecting epoxy resin. About 80 tons were used.

Source (NCE Intl. October'83)

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* Paper for presentation at seminar 3-12-93, Colombo.