XI. SEMINAR FOR TECHNOLOGY TRANSFER

XI. Seminar for Technology Transfer

A seminar was given to the NAPOCOR's engineers for the purpose of the technology transfer as part of this study at the NAPOCOR Training Center on July 28 and 29, 1986.

The subject and the lecturers were:

"Study on the Stability of Dams from Geological and Geotechnical Points of View" by Dr. Haruo Tanaka.

"Maintenance and Control of Fill Dam" by Mr. Yutaka Matsui.

"Investment Planning for Hydro Power Facilities" by Mr. Hayao Adachi.

Participants in the seminar were from the respective departments and sections of NAPOCOR and NIA, and totaled 35. Texts used for the seminar are as attached hereto.

MATERIALS USED FOR THE SEMINAR

1. Study on the Stability of Dams from Geological and Geotechnical Points of View

2. Maintenance and Control for Fill Dam

3. Investment Planning for Hydro Power Facilities

STUDY ON THE STABILITY OF DAMS
FROM GEOLOGICAL AND GEOTECHNICAL POINTS OF VIEW

July, 1986

Dr. Eng. HARUO TANAKA
Engineering Geologist, Registered
Senior Technical Adviser

JICA CALIRAYA DAM
REHABILITATION PROJECT

Study on the Stability of Dams

from Geological and Geotechnical Points of View

CONTENTS

1. Introduction

In terms of construction materials, dams are generally either concrete or fill type. Concrete dams are sub-classified into various types, as follows: gravity, hollow gravity, arch and buttress. Gravity and fill type dams retain reservoir water pressure by their own weight, whereas an arch dam transmits the stresses caused by impounded water to both sides of the valley by the combination of arch and cantilever actions. The thickness of an arch dam tends to be less than that of a gravity dam, with its concrete volume usually about 50 - 70% of that of a gravity dam. However, the stresses within the dam body are greater, therefore the quality of concrete in the arch dam should be superior to that of gravity dam. Furthermore, there are several complicated factors affecting construction of arch dam, such as:

(1) A narrow river gorge must be selected for the arch dam site with geological and topographical conditions good enough to resist arch action. Therefore, sites suitable for arch dams are rather restricted owing to these geological limitations.

(2) Structural and seismic analysis require considerable time.

(3) Complicated form works.

These factors are much less problematical in the case of concrete gravity type or fill type dams and therefore these types of dam have become prevalent in recent years.

Up until 30 to 40 years ago, it was generally considered that a dam could be constructed on any type of foundation rocks without consideration of geological conditions, because dams were few in number and their height was rather low.

Since the Second World War, the construction of large dams has become widespread throughout the world. In such cases the water pressure and stress acting on the foundation rock is big and, accordingly, greater emphasis has been placed on the importance of foundation geology. To-day, it is generally understood that the foundation rock and structure are one body, and the failure of foundation rock is linked to the failure of dam structure.

The late Dr. C. Semenza (Consulting Engineer of Electro Consult Italy) who visited the Kurobe No.4 Arch Dam as a technical consultant said that "a chain play a good function when individual rings have a complete shape and strength, but if one ring should be broken, the chain will immediately loss its function". An adequate understanding of dam foundation geology is one of the essential factors in promoting an appropriate design and ensuring dam stability.

In order to promote a thorough understanding of a dam's foundation geology, geological investigation should be performed with an accuracy which corresponds to the prefeasibility, feasibility and design study stages respectively.

Geological investigation will sometimes continue into the construction stage particularly where unfavourable geological conditions are found in the foundation area in order to promote suitable structural design changes in conformity with the geological defects or to consider suitable countermeasures to overcome such unfavourable geological conditions. The late Dr. C. Semenza, ever said that "when I looked back on the geology of Pieve di Cadore dam (the Pieve River, North Italy), I remember that it was sametime with completion of dam and finish of geological exploration of dam foundation". His recollections well express the direct relation ship between construction works and geological investigations.

2. Geology of Dam Foundation

Gravity or fill type dams can be constructed on almost all kinds of foundation rocks. The numbers and kinds of foundation rocks in Japan is illustrated in Fig. 1. The developments of theories and techniques in all fields of engineering and the activities of dam construction in the world made possible the construction of dams on the so-called inferior geological foundations by means of high quality design and foundation treatment works reinforced by theories and techniques. Obviously, where heavy foundation treatment works are required, construction costs will be high. Therefore it is advantageous to choose a site where the geology is good enough and to require no heavy foundation treatment. In other words, it is no exaggeration to say that the foundation geology is evaluated heavily by the cost of foundation treatment works. In some cases the cost of foundation treatment is almost equal to the cost of construction of the dam body. However, in cases where a dam is considered to have high economic potential for power generation, irrigation, flood control, water supply or navigation, and/or the dam is considered essential for the security of the inhabitants of the downstream area, the plan for dam construction will be progressed, regardless of high cost.

The foundation geology of dam is created by nature and therefore, it is less massive than concrete. Even where the dam site is expected to be a geologically sound, numerous joints, fissures, cracks, bedding planes and fractured layers (these are called "geological separation" hereafter) will be found after removal of over-burden. At a dam site, where geology is expected to be medium, the excavated rocks will be separated into angular blocks or pieces by numerous geological separations. Some of such geological separations will appear yellowish brown coloured by dissemination of limonite and clay lines, while others will be locally altered by weathering into soft rocks.

Examples of the above mentioned rock conditions at dam sites can be found in the Philippines, Indonesia and Japan where the writer has made detailed geological investigations, because those countries belong to the geological structual zone in a recent geological age. To ensure dam stability, design should be revised in conformity with geology, not only during feasibility and design stages but also during construction works. Modern dams are designed and constructed in this way. In other word, there is the geology and design follow the geology, and not the geology follow the design.

Generally, when assessing dam site geology, the expressions "good" or "bad" geology are used. However, it should also be borne in mind whether the geology is "bad" from the view points of lack of stress registance or facility of impounded water leakage. The preliminary selection of dam site, determination of dam type, design modifications and cost estimates of foundation treatments will be formed with above considerations.

3. Geological Investigation of Dam Foundation

There are many published criterias and manuals on dam foundation investigations the detail of which will be omitted in this paper. However, the process and method in each study stage will be summarized below briefly.

3.1. Pre-Feasibility Study

At the pre-feasibility study stage, the proposed dam site area will be selected by a reconnaissance survey considering geology and topography. Then as part of a first step investigation it will be checked whether or not there are serious topographical, geological, lithological and geotechnical defects, by means of few drillings and analysis of aero-photo. For prefeasibility study purposes, 1 : 50,000 to 1 : 100,000 scale maps are commonly used, however, where larger scale maps (for instance 1 : 10,000 to 1 : 20,000) are available, these should obviously be used.

3.2. Feasibility Study

After completion of pre-feasibility geological and topographical studies, the most suitable dam site and the reservoir area are selected. A few drill holes and adits are driven along assumed center line of dam. The drilling cores, ground water levels and geology in the adits are checked and analyzed by geotechnical engineers and geologists. Based on such data the location and depth of further drill holes and adits are determined.

The judgement of dam type (concrete or fill type), feasible height of dam, design of suitable foundation treatment method and estimation of cost of foundation treatment works are discussed among civil engineers, geotechnical engineers and geologists. Frequent discussion of the above problems will be of great benefit in dam construction.

At the discussion of the geology of dam site, the rock rating is required to establish a common standard for evaluation of dam foundation geology. The rating will be available to imagine the general aspect of whole rock mass from data of rock tests (or observation) in adits and/or drillings and to compare lithological conditions and foundation treatment works in the existing dam sites. The study on grad up of dam foundation rocks rating is essential, however, it must be kept in mind that the rating should be too detail to applicate for an actual field investigation. In other words it must have a resonable aptitude from engineering point of view. The R.Q.D. (Rock Quarity Designation system - Deere et al 1967) is applied for the rating of drilling cores, and the rock qualities are deginated 0 - 25%: very poor, 25-50%: poor, 50 - 75%: fair, 75 - 90%: good, 90 - 100%: excellent (Percentage recovery 100 m/m length) respectively.

The rating of foundation rocks was also discussed in Japan and many ratings were proposed. Among these ratings TABLE-1 is a writer's rating and TABLE-2 is that of actually used and authorized by the member of BOC (Board of Consultants) for construction of the Saguling (97 m in hight) and the Cirata dam (115 m in hight) in Indonesia.

The ratings in TABLE-1 and TABLE-2 show those converted from the hard consolidated rocks to the medium consolidated rocks. Dr. Iida who had been working for a long time in the Civil Engineering Laboratory of the Ministry of Construction in Japan as a chief of dam section and director of the laboratory mentioned in his book named "Dam Design" (1980) that "The rock ratings proposed by Central Research Institute of Electric Power Industry (TABLE-1) and the Civil Engineering Laboratory of the Ministry of Construction in Japan are useful for evaluation of dam foundation rock generally. However, in case of individual dam site, there are sometimes other elements which master a mechanical properties of dam foundation rocks severely. Therefore sometimes it is required to make a small reorganization of rock rating taking into account of the specified rock properties and the data of rock institute tests."

Judging from above discussions, the rating of rocks is not always absolute and world wide one. The reorganization of rock rating of foundation rocks should be carried out for each individual dam site taking into account of specific site geology with the approval of experienced engineering geologists and geotechnical engineers.

The geological reconnaissance survey should be carried out more precisely than in the prefeasibility stage with 1/500 to 1/1,000 scale topographical map. The geological maps and sections made in Pre-feasibility study stage should be revised by data from the drill holes and adits excuted in this study stage. These maps and sections will be available to determine the location of further drill holes or adits and to change design of dam and planning of foundation treatments. Seismic prospecting is executed at this stage as a supplement to other geological investigation works and geological reconnaissance. The number and location of measurement lines will be determined after chicking and analysing the results of drill holes and adits inspection.

3.3. Engineering Stage Study

Engineering stage investigations proceed after the construction of dam has been framed by the feasibility study investigations. At this stage, investigation work will concentrate on various tests to clarify numerically the physical properties of rocks. The data from these tests will be used to establish the basic dam design and foundation treatment works. The number of drill holes and addits will be increased and will be distributed not only along the central axis of dam but also around the dam foundation including the downstream valley wall.

The in-situ investigation work to determine the rock's physical properties is mostly performed in the field. It includes Jack test (measurement of elasticity coefficient and deformability of rocks), Block shear test (Measurement of cohesion and friction angle), Permeability Tests, Measurements of velocity of elastic wave, of underground water level, and of movement of landslide and Grouting test. In addition to the above tests, uniaxial compression test, microscopic examination and chemical analysis of rock specimens are performed accordingly.

i. Jack Test

This test is performed in the adit. Put a circular loading steel plate on the smoothly excavated rock surface at the floor of the adit, and impose a vertical load by oil jack, measuring the load and amount of deformation of rocks which correspond to the given load. The elastic modulus or deformability of rocks will be calculated from analysis of load deformation curve.

ii. Block Shear Test

This test is also performed in the adit. Put a concrete block on the smoothly excavated rock surface at the floor of the adit, and impose a vertical load by oil jack while lateral load (shear load) is imposed by horizontal oil jack, and measure the vertical and horizontal loads at the instant the concrete block begins to move. One set of the test uses three blocks in order to obtain a curve which determine ϕ and C. The shearing strength which is shown by coefficient and shearing modulus will be calculated from the analysis of testing figure. Instead of concrete block, cube rock blocks consisting of in-situ rocks cutting out by smooth blasting or rock saw are used for in-situ shearing strength test. The principle of the rock block test is similar to that of the block shear test, however it involves the high cost of cutting out the cubic rock blocks and, therefore, it is seldom used these days.

iii. Permeability Test

The in-situ permeability test of the foundation rock mass is performed in the drill holes which are arranged in approx. 30 m spacings along the proposed center line of the dam. The drilling depth is almost equal or half to the height of dam. Water will be forced into the hole and the quantity of water absorption corresponding to the injections pressure at every five meter stage from top to bottom of the hole is measured. The amount of absorption of water with 10 fkg/cm^2 injection pressure is expressed by l/min/meter and is called "Lugeon value". This value should be described in drilling logs and topographical or geological sections of dam central axis. Thus the underground distribution of Lugeon value (i.e. permeable conditions) is understood, and will provide useful data to establish an appropriate depth and spacing for curtain and consolidation grouting.

iv. Measurement of Velocity of Elastic Wave of Rock

The aim of this measurement is to ascertain the physical properties of rock mass generally. They will be represented by the velocity of seismic wave. The mode of occurrence of geological separations such as joints, cracks, fissures, fractures and faults in the foundation rocks will be reflected in the velocity of seismic waves propagated on the rock mass. The figures for velocity do not absolutely express the geological condition of the rock foundation but are useful as a general guide for evaluation of rock mass and for the comparison of the dam site rock properties with others. This method is also used for evaluation of grouted or PS anchored rock masses. This measurement carried out by using a small amount of explosive to create a low level seismic wave in the adit. This wave is received by electric seismograph setting in the other adit. The distance of epicenter and seismograph can be measured, and the propagating time between epicenter and seismograph is measured by a seismographic recording device. Then the velocity of elastic wave propagate between two adits (or in a adit) is easily calculated from a time-distance curve. For the ordinary purpose of measuring the velocity of rock mass, the longitudinal wave is used, the lateral wave is used in special cases only.

v. Measurement of Underground Water Level

The water level at a dam site is lowest in the river bottom and becomes higher in confirmable with topography at both banks of the river. The ground water level is commonly higher than the high water level of the reservoir in both banks, however occasionally, it is lower than the high water level at the location of far away from abutment. In this case, there is a possibility of reservoir water leakage and therefore, the cause of such low ground water level must be investigated carefully and precisely in order to make a plan of leakage prevention works. The ground water level of both downstream abutments should also be investigated by drilling in order to compare the ground water level before and after impounding reservoir, because if there is leakage of water through the curtain grouting after impounding, the ground water level of the drill holes at downstream mountain slope will rise and it will become a guidance of the pore pressure increment in the downstream mountain rock mass. The increment of pore pressure would tend to cause sliding on the downstream mountain slope along the geological separations, giving rise to possible dam failure. The ground water level in the drill holes changes seasonably, and therefore, it must be measured throughout the year by an automatic recording device (piezometer float etc.).

vi. Measurement of Movement of Landslide Area

If potential landslides are found near the dam site by reconnaissance survey, their mode of movement should be measured by means of an automatic recording instrument (The inclinometer, strain gauge in the drill hole, and/or survey stake) in order to predict the sliding movement.

vii. Grouting Test

There are two types of grouting, namely consolidation (blanket) and curtain grouting. The foundation rocks of dam area are treated as an elastic body for the design of dam, however there are numerous cracks, fissures, fractures and faults caused by crustal movement, and these geological separations tend to lose their coherence. The purpose of consolidation grouting is to fill and fasten these loose geological separations by injection of cement in order to restore, as far as possible, the properties of the foundation rocks to those of an elastic body. In the case of a concrete dam consolidation grouting is generally executed over the whole area of dam body. On the other hand, in the case of fill dam it is generally limited to the area beneath the core or facing concrete, except in cases where there are serious geological conditions beneath the dam body. The drill-hole spacing for consolidation grouting is commonly 3m - 5m with zigzag pattern and less than 12m in depth. The consolidation grout test is carried out in order to make a rough estimate of an appropriate grout hole spacing, of amount of cement absoption per hole. The consolidation grout test is generally carried out in a specified area of the dam foundation. The following three methods are adopted to the check the grouting results namely; (1) visual inspection, (2) measurement of decrease of amount of injectied pressurized water in a newly drilled check hole, and (3) measurement of increase of velocity of seismic wave. Of these, pressurized water injection is the most reliable method.

The geological separations not only cause loosening foundation rocks but also become a cause of water leakage because, such geological separations commonly develop even in the deeper portion of foundation rocks. These leakages are prevented by curtain grouting.

Curtain grouting is an injection of cement milk through bore holes drilled almost parallel to the axis of dam in a two row zigzag pattern. The drill hole spacing is commonly 3 m and the depth is generally half or as same as the dam height.

The curtain grouting test is executed for the purpose of checking of (1) effectiveness of grouting, (2) determination of minimum spacing of grout holes and (3) assessment of grouting pressure and grout take. The location of grouting test is desirable as close to the proposed axis of dam as possible, and the test should be performed in the deeper portion than estimated rock excavation line. The grouting test holes are arranged in triangular pattern on a concrete platform installed in a specific area. (FIG.-2). The maximum spacing of hole is 6 m (these holes are called primary holes) and the spacing of succeeding secondary and tertiary holes is so arranged to be half of the preceeding spacing. Namely, the spacing of holes to each triangle will be 6 m (Primary hole), 3 m (Secondary hole) and 1.5 m (tertiary hole) respectively in final. The check hole is drilled at the center of the triangle. The grouting test is performed in 5 meter stage from top to bottom

in the primary, the secondary and the tertiary hole respectively. The procedure of grout test is as follows:
The test is started from primary hole. At first, a hole drilled 5 m in depth (No.1 Section) from the ground surface. Prior to grout injection of cement milk into No.1 section, permeability of the rock is checked by injection of pressurized water. The initial injection pressure of water will be determined in accordance with geological conditions.

After the above permeability test, the cement milk is injected into the hole. In this case the mix ratio of water and cement and injection pressure of cement milk will be determined with reference to the results of pressure water test and experiences. When the 5m section grouting test is finished in each primary holes, the result of grouting performance is checked by the permeability test in the 5m deep check hole at the center of triangle. After the checking of permeability at the central check hole, the primary holes are extended till 10m in depth as No.2 section. The permeability test, grouting and after grouting permeability check are performed with same procedure as in No.1 section. The No.3 (15 m in depth), No.4 (20 m in depth) and descendent sections are performed and checked with same procedure as mentioned above.

When the tests in the primary holes are finished, the permeability and grouting tests are performed in secondary holes with same procedures as in primary holes.

The permeability and grouting tests are performed in tertiary holes with same procedure as primary and secondary holes.

Through these tests proper spacing of grout holes, mix ratio of water and cement and injection pressure will be determined used in actual curtain grouting works.

As mentioned above grouting test is expensive and time consuming, moreover the test area is substantially apart from actual grouting area to judge an actual of curtain grouting. Therefore, some engineers are of the opinion that it is more effective to perform the curtain grout testing in the very curtain grouting area parallel to the actual grouting works in the construction stage.

4. Foundation Treatment

Generally there are numerous geological separations in the dam foundation rocks. Recently dam sites with good geology have been increasingly rare and accordingly dams must be constructed on foundations of inferior geology. Consequently, the treatment and reinforcement works on the geological separations in the dam foundation have become an increasingly important part of dams construction. Such works are known as foundation treatment works, and consist of (1) grouting, (2) installation of drain holes, (3) replacement by concrete, and (4) reinforcement by anchor bar.

Grouting is the common and worldwide method to improve foundation rocks. Grouting is divided into two methods, one is consolidation grouting and the other is curtain grouting. (cf grouting test) For both consolidation and curtain grouting, the injected materials are principally water and cement, however when there too much grout absorption occurs, sometimes sand is added to the grout. Where tremendous quantities of spring water make cement grouting impossible, chemical grouting is applied. However, the strength and durability of this material is inferior to the cement and it's cost is higher than cement, therefore, it is not commonly used except in special cases. Since the acrylamide group chemical materials, named AM-9, were developed for grouting in USA, several kinds of chemical injections have been developed and used around the world. However, since 1974 the use of acrylamide group chemical materials has been prohibited in Japan by provisional instruction of the Ministry of Construction, due to underground water pollution problems except in cases of urgent necessity. Nowadays water glass group injection which carries no risk of population is used instead of acrylamide group.

Regarding the foundations of concrete dams there are some discussion on the time of execution of consolidation grouting. The question is whether it is preferable to perform injection before or after concreting. The advantage of grouting before concreting is the convenience of being able to confirm with the naked eye the mode of cement intrusion into geological separation. The disadvantage is that high pressure cannot be used. On the other hand, the advantage of grouting after 2 or 3 lifts of concrete is that moderately high grout pressure can be used in order to securely fasten the geological separations. With this method, leakage of milk over the rock surface will also be prevented. The advantage of both methods should be determined by the discussion among civil engineers and geologists.

Whatever thickness of grouting is executed water leakage through the grout curtain is inevitable. Large leakage will cause not only uneconomic wastage of reservoir water but also safety problems to the dam foundation from piping. Unless sufficient leakage protection works are executed, large leakage through foundation rocks tends to occur in rocks such as limestone, lava flow and poorly consolidated volcanic breccia, sand and ash.

Water passing through the grout curtain will usually be subject to high water pressure (corresponding to the depth of reservoir), therefore imposing an uplift force on the dam body. It will usually be drain by drainage bore holes arranged in a line few meters downstream of grout curtain line and removed by pumping. If the arrangement of the drain holes or performance of the grout curtain are insufficient, the leakage water can intrude into the downstream mountain on both banks, and rise the ground water level causing an increase of pore pressure and decrease of friction strength of rocks on the mountain slope. This way eventually cause sliding or collapse of the mountain slope adjacent to the dam abutments. If an abnormal increase of the ground water level in the inspection bore holes in the downstream mountain slope of dam is detected, it must be studied whether this is due to rainfall or reservoir water leakage. If it becomes apparent that the rise of the groundwater level originates from leakage, countermeasures such as driving horizontal drillings or drainage adits to draw down ground water level must be taken.

In short it is a fundamental rule that underground water running through geological separations in the foundation rock mass, should firstly be prevented by the grout curtain, and then water leaking through the grout curtain should be drained as soon as possible by the drainage system. As to leakage water downstream of the dam, pressure, quantity, turbidity, temperature, and pH must be measured at suitable intervals, and consideration given as to whether or not the leakage is such as to endanger the security of dam. Among above elements, the measurement of turbidity and quantity of leakage water is most useful for the judgement of the security of foundation, because if some piping action should be taking place, turbidity and quantity of leak water will increase step by step.

To prevent leakage, additional grouting to the assumed pass of the leakage water is the most likely measure. However if the condition of the dam seems serious, drawdown of reservoir water level must be considered, because the stress to the dam by water will decrease rapidly by drawdown of reservoir level.

Regarding the faults and fractures appearing on the excavated rock surface, narrow ones will be treated by grouting, but others which are over 50 cm in width will become a cause of local subsidence and/or leakage. These faults or fractures should be excavated in some depth and backfilled with plain concrete which is considered as artifical rock. As to the depth of replacement by concrete, there are many theoritical and/or empirical formula based on the data of in-situ rock elastic modulus and shear strength tests. In most cases the depth is determined through discussion between civil engineers and geologist.

In some cases a thick flat or arch shaped concrete slab is placed on the backfilled weak zone like a bridge. This method, being akin to dentists' remedial methods, is therefore known as dental work. However, this method is not used because deep replacement of a weak zone will injure the adjacent intact rocks and moreover the inner arch action arising in the thick backfill concrete will to open the faults or fractures bringing undesirable effects to the foundation. For the same reason the covering method with arch shape concrete bridge was not also prevailed recently. Instead of mass backfill concrete, the method of covering the weak zone with flat reinforced concrete has preveiled. In such cases the removal of the weak zone is limited only to the shallow portion and will not injure the adjacent intact rocks. As an alternative to the flat reinforced concrete method, sometimes mesh steel bars are set in the base of dam concrete in order to resist unequal subsidence of base rock and to prevent the elongation of cracks in dam concrete.

To prevent sliding of the rock masses along faults and fractures, reinforced concrete keys branched from the main trench for back-fill concrete are rectangularly embedded into the intact rocks on both sides of weak zone. The frictional function of these keys will prevent relative movement of rocks along weak zone which is liable to move by the stress of dam. The numbers of concrete keys are determined by the magnitude of the stress transmitted from dam body.

If the key method mentioned above is judged not to be available or not to be suitable from the mode of occurrence of weak zone, binding both sides of intact rocks across the weak zone by PS-anchor is adopted. The method of PS-anchor binding is to drill bore holes (ϕ≒100 - 120 m/m) from the surface across fault or fracture, and then insert a steel bar or a strand wire into the bore holes, and the end of steel bar or strand wire is fixed about 5 m in length in the intact rock mass by injected cement milk. The head of the bar or wire is fixed by a square concrete washer on the rock surface and the steel bar or wire is tensioned by oil jack. By the reaction of this tension force, sliding of the rock mass is prevented. The force for tensioning is determined in accordance with the data of in-situ rock tests on the weak zone and the total force of sliding of rock masses. Tension rock bolts are used instead of PS-anchors when the required stress to prevent sliding is small. The above foundation treatment or reinforcement methods have recently been applied in improvement works on large dams. These methods must be designed based on the theoritical examination, utilizing data given by the investigations, measurements and testings. Among these data, permeability, elastic modulus, deformability, shearing strength (including gauge material) are most important.

The uniaxial compressive strength test of rock pieces is performed to ascertain the strength of rock pieces rather than to establish the strength of rock masses. For this test, cubic samples (approx. 10 x 10 x 10 cm) taken from the adits or selected samples from the drilling cores will be used. The core samples will be reformed at the laboratory, namely, the height of specimen will be twice of diameter of cores. A vertical load is imposed on the samples with a testing machine and the load of the failure of rock sample is measured. When the core samples are drilled, the cores of inferior geology portions should not be recovered as cores suitable for use as a test piece. Therefore, the test data is liable to indicate the strength of good rocks, and then it is somewhat ambigious to determine the strength of rock mass. However, the strength distribution map drawn in the cross section of dam site with the core test data in various depths of drillings will become a good reference to evaluate the difference of rock strength and will be useful for general conception on the rock strength beneath the dam body.

The name of foundation rocks is determined by visual inspection with a magnifying glass. In some cases, it is difficult to determine the rock name by the above method and sometimes it is required to determine the name exactly. In such a case, it is determined under a microscope with thin section of rocks. As to the clay minerals in the gouge materials, the kind of clay mineral is determined by X ray analysis, and if the potential swelling minerals such as montmorillonite, kaolinite, and irite etc. are found in the minerals, a swelling test will be performed in order to ascertain the degree of swelling in a saturated condition. The results of swelling test will be used for designing remedial treatment of faults in the foundation rocks of the dam.

Concrete alkali-aggregate reaction is mainly caused by the chemical reaction between alkali contained in cement and silicate components of aggregate, and by this action the strength of concrete weaken resulting in a development of cracks in the concrete. Alkali-aggregate reaction is likely to occur when Liparite (Rhyolite), Dacite, Andesite, andestic or liparitic tuff and a kind of phillte are used as aggregate. Where the dam aggregate or foundation rocks have a relation to these rocks, the concrete of dam may be damaged by alkali reaction and, therefore, the alkali-reaction test of rocks is required. There are some instances that the Pyrite group minerals are contained in the foundation rocks and these minerals sometimes change into sulfuric acid when contact with water and air. The sulfuric acid thus produced will lead to the corrosion of concrete or reinforcement bars. If Pyrite group minerals are found in the foundation rocks a quantitative analysis of Pyrite group minerals is necessary in order to establish whether the amount of Pyrite group is harmful to the structure of dam body or not.

5. Geology of Reservoir Area

There will be many kinds of geology in the reservoir area, therefore it is difficult to lump together the geology of reservoir. However, from view point of influence of geology to the function of reservoir, the investigation of potentiality of leakage of water, landslides, or land collapses will be most important. As to leakage of water, the existence of limestone caves or fissures, Karst, volcanic caves and fissures unconsolidated volcanic ash, sand and detritus, and such soluble materials as rock salt and gypsum will become a main problem. It is well known that there are sometimes a caves or large fissures, doline or underground rives in a limestone area, and such geology can cause major leakages. In a limestone area there is a specific topography called Karst, where the distribution of limestone will easily be distinguished from ordinary reservoir area composed of non soluble rocks. The volcanic detritus such as volcanic breccia, sand and ash derived from recent volcano are generally unconsolidated and liable to form a leakage pass. Moreover, the leakage water through such deposits will sometimes transport fine materials which compose the deposits and enlarge a route of leakage pass by piping. In Japan there are some reservoirs under construction where a large amount of leakage prevention works are executing for protection in the unconsolidated volcanic detritus area. That is, as countermeasures against leakage, a large concrete facing has been constructed on the outcrop of these rocks in the reservoir area.

When major faults or fractures pass through the reservoir and extend to the outer rim the reservoir or when the ridge of reservoir rim is so thin as to require reinforcement by dyke or facing, the geological defects in this area must be investigated by reconnaissance survey drilling and/or adit excavation.

Photo-geological analysis by aero-photography of the reservoir area will be a good supplement to a reconnaissance survey to find out the above-mentioned geology. The necessity for leakage prevention works in these areas will be determined by the permeability coefficient and the difference of elevation between the inlet and outlet of the leakage route. The permeability coefficient will be measured by an in-situ permeability test and the presumed leakage pass length will be measured by surveying. Using these data, the velocity and quantity of leakage water at the outlet will be calculated theoretically. In this case, the most important problem for the security of the reservoir is the velocity of leakage water at the presumed outlet. If the velocity of leakage water at the outlet is less than the velocity at which no fine material occurring in the leakage route can be transported, then the leakage is not harmful for the stability of reservoir. In short the security of reservoir mostly depends upon the length of leakage pass (creeping distance).

The fluctuation of the reservoir water level will cause fluctuation of the ground water level, and this will cause an increase and decrease of pore pressure in the mountain slopes around the reservoir rim. As a natural consequence, the mountain slopes will loosen, and rock masses or brocks will slide down into the reservoir. Sliding is liable to occur along daylight structure of geological separations, but sometimes will be occured by swelling of gauge in the fault or fractured zone.

A well-known major slide occurred immediately upstream of the Vajont dam (H=262 m) in North Italy. In this slide, 200 - 300 milion cubic meter rockmasses slide in the middle of the night of Oct. 1963 with the loss of 2,600 lives and considerable property. The slide occurred along fractured layers inclining towards the reservoir and extendeding 700 m long above the reservoir level on the mountain slope. The slide moved with velocity of 30 m/sec. and the duration time of sliding was said to be about 5 min.

The location, the relative height of the summit of sliding mass which has a serious relation to potential energy, and the volume of sliding mass must be carefully taken into account in order to prevent sliding, even in an ordinary reservoir area.

If sliding occurs far from the dam, pulse shape hydraulic bores induced by the sudden plunge of the sliding mass into the reservoir will, owing to its long propagation pass, lose it's energy, becoming a gentle wave before reaching to the dam. If, however such a slide occurs near the dam, the consequences will be quite different with a considerable quantity of reservoir water overflowing the dam crest at high velocity, caused by high hydraulic bores, causing great damage to the dam itself and to persons and properties downstream. When the land slide material slides gently into the reservoir at low speed, the increase in water level is gradual and the water will gently overflow the dam crest without major damage. The volume of slide mass will also have a direct relation to the volume of overflow. In other words, if slide mass is small, it will not cause great damage to the dam or the downstream inhabitants, but large mass sliding may cause considerable damage. The instance of Vojint Dam is one extreme example.

The prediction of potential sliding areas is one of the most important items in reservoir construction. Location, relative height of summit of a sliding mass, and the possible area and course of sliding must be predicted. For this purpose, geological reconnaissance, drillings, addits and ground water level measurement by piezometer, surface water quantity measurement by weir and monitoring of rockmass movement by inclinometer or other means will be essential at any area where sliding is foreseen. Moreover, prediction of increase in reservoir water level by calculations assuming sudden occurrence of sliding is essential. Slide prevention work should be also studied in order to prevent damage to the local inhabitants and their properties. Small scale rock falls or collapses sometimes occur in any reservoir. The sliding or collapses around the reservoir rim along newly constructed roads is common, and is sometimes accompanied by trafic jams, but such small slidings do not threaten the security of reservoir.

6. Sedimentation of Reservoir

Over many years the rocks on the mountain slope change into soil by physical and chemical weathering, and these weathered materials (sand and gravel etc.) are transported to the sea through ravines, tributaries, and main rivers. The construction of reservoir breaks the natural equilibrium of river flow, and so upon completion of the reservoir, the river water loses its transportation energy and deposit sand and gravel in the reservoir in accordance with the rule of "At the location in the river where velocity and/or volume of running water decreases, sedimentation of transported materials will take place". Sedimentation occurs to the extent that sand and gravel are supplied from catchment area of the reservoir.

Finally, sedimentation tends to fill the reservoir and lose its function below the crest of solid dam. The measures to prevent sedimentation are (1) Installation of scouring gates at the dam, (2) Construction of a Sabo dam (check dam) in the upstream area of reservoir, (3) Dredging of sediment material. However, each of these countermeasures has advantages and disadvantages and none of them fully overcomes the sedimentation problem. In order to avoid these serious conditions of reservoir, the assumed annual volume of sedimentation is forcasted with several formula which are studied with reference to geology, topography, precipitation and hydrology in the catchment area. In general, the assumed total volume is added to the planned reservoir capacity with increase of dam height. The factors principally related to volume of sedimentation are annual precipitation, intensity of rainfall, steepness of mountain slope, altitude of mountain, density of vegetation, total area of collapses in the catchment area and it seems to be difficult to deline the main factors of sedimentation. However recently, the annual amount of sedimentation is predicted by the analysis of the data of measurement of bed load and of suspended load in the duration of both normal and flood water. Morever, the volume of sedimentation in existing reservoirs is also taken into account for the final prediction of volume of sedimentation in the reservoir. The dredging of sedimented sand and gravel in order to restore full reservoir volume was formerly considered uneconomical owing to the high transportation cost and the difficulty of acquisition of a spoil area. However, recently, the collection of natural sand and gravel from ordinally river bed is prohibited for the protection of foots of dike and/or of pier of bridge, and then the collection of sedimentary sand and gravel in the reservoir become highlighted in order to fulfill the demand of aggregate for construction works in Japan. Dredging is generally commenced from the head of the backwater where the depth is shallow. Sedimentation spreads from the head of the backwater, so dredging here is a good means of preventing the advance of sedimentation to the dam area.

7. Active Faults and Seismicity Around Dam Site

An active fault is defined as one which has moved repeatedly during the "recent geological age", and accordingly is expected to move in near future (in geological meaning). The term "recent geological age" is usually used to mean the Quarternay Period (from about two million years ago to the present). Generally speaking, it is well known that surface faulting occurs at the time of large destructive earthquakes. Faults with evidence of historical displacement are called earthquake faults (surface earthquake faults). On land in Japan, more than twenty earthquake faults are known. For example, the Neodani Fault associated with the Nohbi Earthquake (1981) and the Gomura Fault associated with the Kita-Tango Earthquake (1927) are famous all over the world. It is an important fact that most of these earthquake faults appeared along the fault zone which had been recognized as active fault from many geological evidences. This also indicates that any active faults have a potentiality to become surface earthquake fault accompanying the large destructive earthquakes in future.

However, it is a fact that there are faults which occurred during the quarternay period and yet have no evidence of subsequent movement. Moreover, the above-mentioned scale of "recent geological age" and the term "near future" are very difficult to understand from the engineering sense. Therefore, if the word of "near future" should applied in terms of the life of civil engineering structures, the word must be defined as "a necessary period to insure the security of the structures". Also, the term "active fault" is liable to mislead engineers as if the fault is always moving as active volcano.

Recently this problem was discussed among engineers and geologists in the committee organized by Ministry of Construction of Japan, and usage of the term of "Quarternay fault" is proposed in place of "active fault".

If active faults (Quarternay faults) are anticipated in or around a proposed project area, various investigations should be undertaken to determine their precise locations, length, width, and degree of activity. Practical methods for such investigations are as follows.

(i) reading and analyzing aerial photographs to find out lineaments vertifal displacement of terrace surface and/or horizontal displacement of river course, topographically

(ii) field investigation by geological reconnaissance survey

(iii) driving drill holes and/or adits along fault lines assumed from lineaments and/or terrace surface or river cause displacement.

iv) evaluation of degree of activity by trenching method on fault line.

If any displacements along the fault in the younger quarternary strate is recognized, it is required to estimate the geologic age of the displaced younger strata. There are many instances in terrace deposit that lower (older) strata is displaced but upper (newer) one is not displaced. In these cases, estimation of the geological age is required not only to lower strata but also to upper strata. The measurement or correlation methods of age of younger strate are (1) Radio carbon dating (C14 method), (2) Potassium-Argon method, (3) Tephrochronological method (correlation of Tephra), (4) Pollen analysis, (5) Surface of Terrace or lava flow correlation method, (6) Red soil correlation method, (7) Palaeomgnetism method. Among these methods the C14 method is the most reliable in engineering point of view and widely used. This method is to measure the percentage of C14 remaining in past life remains (Fossile) buried in the strata and to determine absolute date by year. The upper limit of absolute data which can be measured by this method is about 30,000 - 35,000 years. Beside above, the age of fault itself is directly measured by the analysis of minerals contain in fault gauge. They are (1) Fission Track method, (2) Electric spin resonance method, (3) Analysis of surface erosion of quartz. However, the evaluation of these methods are not fixed generally, for the engineering purpose of dating of age of faults in the present, therefore, these methods are utilized when the younger strata do not contain fossils or the above mineral analytical methods are only a way to determine the age of faults.

It is empirically known that the maximum magnitude of earthquake which is induced by the displacement of surface of earth have a good relation with length of fault. There are several experimental formulae about the scale of earthquake and length of active fault (quarternary fault), and most of the formulae have following forms.*

$$\log L = KM-C$$

L : length of active (quarternary) fault

M : max magnitude of earthquake

K.C : constant

Also, there are following empirical formulae between amount of maximum displacement taken place by earthquake and maximum magnitude.

$$\log D = k'M-C$$

D : max. displacement

M : max. magnitude of earthquake

k'C' : constant

By the above equation maximum magnitude (M) will be calculated from the length (L) of active fault (quarternary fault) and the maximum displacement (D) will be calculated from the maximum magnitude (M).

As explained above the length (L) of active fault (quarternary fault) has a important meaning for design of structure.

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* MATSUDA, T. (1977) : Estimation of Future Destructive Earthquake from Active Faults on land in Japan, Jour. of Physics of the earth Vol.25 (Supplement) 251-260.

For the evaluation of potentiality of moving of the active fault (quarternary fault), the history of movement of the active fault (quaternary fault) must be carefully taken into account, because it is said that the active fault (quaternary fault) which has evidence of recent movement has a high potentiality of moving in future. Therefore, great care must be taken when faults which have evidence of moving in historical earthquake or in the Neolithic age pass near a dam site.

The age of active faults older than historical and Neolithic age is determined by C14 method and/or other geological correlation methods. However, it is said that the life of concrete dam is about 100 years and this life age will be only one instance in the geological history which is counted with a unit of several ten or hundred thousand years. Therefore, some engineers greatly doubt the possibility of movement of the faults in such a short duration as life of dam even if, the faults has a evidence of movement in several ten or hundred thousand years before present taking into account the frequency of activity of the faults. In accordance with a study of the historical movement of active faults (quaternary fault) in Japan, the fault moved five times within 21,000 years and total amount of movement reached 8.5 m, but the fault moved about 8 m within a half of moving duration and 42 cm moved in remainder duration, this fact will be one of the route of above discussion.

8. Earthquake Resistance of Dam

About 90% of concrete dams in Japan are concrete gravity, and there are no instances of dam failure caused by earthquakes. This fact shows empirically that the concrete gravity dams have a high safety against earthquakes. In Japan it is indicated by regulation of Ministry of Construction that the dams must have over 4.0 in safety factor again worst conditions. Dams have been designed in accordance with this regulation and therefore they have a high safety against earthquake vibration.

Generally speaking, dams have been constructed on the well consolidated rocks older than the quarternary era and the weak zones in the rock masses are improved by foundation treatments.

Damage to structures mainly depends upon the geological conditions of the foundations, a fact prove in the recent earthquakes (MIYAGI OKI 1978 M=7.4 and KANTO 1923 M=7.9) in Japan. Namely the damage to houses or structures was greater in the city centre areas composed of quaternary deposits than to those in hiily areas composed of younger tertiary formations.

Dr. Iida describe the results of investigation in his book* about damage to 17 concrete dams in the area bound with a radius of 300 km from the epicenter of the NIIGATA earthquake (1964 M=7.5). According to his discription, one hollow gravity dam was damaged by the development of 70 cm long cracks and a few dams were damaged by increase of a small amount of leakage, however, there was no damage having a serious influence on dam stability. He also describe the results of investigation on 62 concrete dams (including 3 arch dams and 2 hollow gravity dams) on the occasion of the Miyagioki earthquake in TOHOKU district. According to this discription 61 dams were not damaged and one dam damaged by temporary leakage.

K.D. Hanson and L.H. Roehm** reported the response of the 17 concrete dams (including gravity, arch and buttress dam) in nine countries have been subject to significant vibration difined as ground shaking in excess of 0.10g.

The result of the studies are shown in Table-3.

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* IIDA : Design of dam. New system of Civil Engineer, GIHODO TOKYO JAPAN, 1980

** K.D. Hansen and L.H. Roehm: The response of concrete dams to earthquake. Water power and construction April 1979

From above Table-3 it can be easily understood that damage to the concrete dam sometimes occurs at the section near the crest of dam where the section of dam body suddenly becomes thin or occurs by separation of block joints without causing serious damage to the dam body. On the other hand there are many instances of damage in earth dam in Japan, although it is difficult to discuss earthquake stability of them generally, because the date of completion, the construction methods, the materials, especially compaction methods, are not clear in individual dams.

Foundation geology is a big factor in earthquake resistance of earth dams and the dams built on formations older than quarternary era are less damaged than these on the quarternary formation composed of sand, clay, or silt mixtures thereof.

The Table-4* show the damage to earth dams investigated by Mr. Moriya** on the occasion of TOKACHIOKI earthquake (1968)

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* The instances of damaged dam in Japan are mainly low earth dams (more or less 10 in height) which are called irrigation poinds.

** M. Moriya: About damage of earth dam by earthquake, large dam No.48 1969.

Table - 4 Mode of damage of earth dams on the occasion of TOKACHIOKI earthquake

Height of dam / Mode of damage by earthquake	less than 10 m		over 10 m	
	Nos. of damage	%	Nos. of damage	%
Failed by sliding	9	10.5	1	12.5
Sliding of upstream face	21	24.7	4	50.0
Sliding of downstream face	10	11.7	0	0
Sliding both upstream and downstream faces	4	4.7	0	0
Cracks on dam body	24	28.2	1	12.5
Subsidence of dam body	7	8.2	1	12.5
Break of intake structure	22	25.8	2	25.0

(After M. Moriya)

Note: Nos. of investigation 93 : Less than 10 m in height : 85
More than 10 m in height : 8

The recent zone type fill dams which are built on a good foundation geology with good compaction and excellect selection of materials have an inherently high resistance to strong earthquakes. The Table-5 shows seismic performance of existing rockfill dams in the world.

The construction of a reservoir changes the water pressure beneath the reservoir and consequently the stress balance of the intact rock formations beneath reservoir changes. This allows previously stressed rock to slip, mostly on existing fractures and generate earth motion. There are many instances in the world of reservoir induced microearthquake. It is reported that such earthquakes have a tendency to occur in or around a reservoir where the total storage capacity is large, more than 1,000 million ton and the dam

height exceeds 100 m. The magnitudes of reservoir induced seismicity are predominantly small, less than Richter 3. Such earthquakes are rarely felt by humans and do no damage. Occasionally they may exceed magnitude three and are large enough to be felt and sometimes even produce minor structural damage. In two or three dams the largest magnitudes have been five or more causing some damage.

A few microearthquake measurement stations should be installed for the purpose of the measurement of microearthquake activity in the projected area prior to dam construction in order to obtain information on micro seismic activity before impounding as well as possible reservoir induced seismicity.

In the instance of the Cirata and Saguling Projects in Indonesia, the microearthquake station network consists of 6 networks, each available for the hypocenter processing, and magnitude estimation (Fig. 3). The seismic monitoring by these networks commenced in February 1985 and has been continuing. The frequency of microearthquakes has increased since commencement of impounding.

The processing of hypocenter will be available for the judgement of an active fault (quarternary fault), that is to say, if hypocenters arranged along on a line which was assumed as active by geological and geotechnical investigations, then the fault may be judged as active.

The micro-earthquake montoring in the projected area should be continued over a long period in order to ascertain the relationship between microearthquakes and reservoir level, underground water level, landslides, and active fault (quarternary fault etc.).

9. Conclusion

Dam construction takes a considerable time from planning to the completion, through prefeasibility, feasibility, design and construction stages. In any stage, planning and design must follow the geological conditions.

The engineer, geologist and geotechnical engineers must ensure a close exchange of information and always constructively discuss the promotion of project. For discussion to proceed smoothly, the engineer must understand the geology and the geologist and geotechnical engineer must keep fully in mind the design and mechanical affairs of the structures. With such an atitude of combined efforts of engineers and using the methods of investigations, tests, measurements, and executions previously explained in this paper, an economical and secure dam will be constructed. After completion of the dam, monitoring and surveillance of the dam, reservoir and the neighbouring area are essential to maintain the stability of the structures.

FIG. 1 Histogram of the Kinds of Foundation of Rocks of Dams for Multipurpose and Hydroelectric Power Projects

(713 dams in Japan)

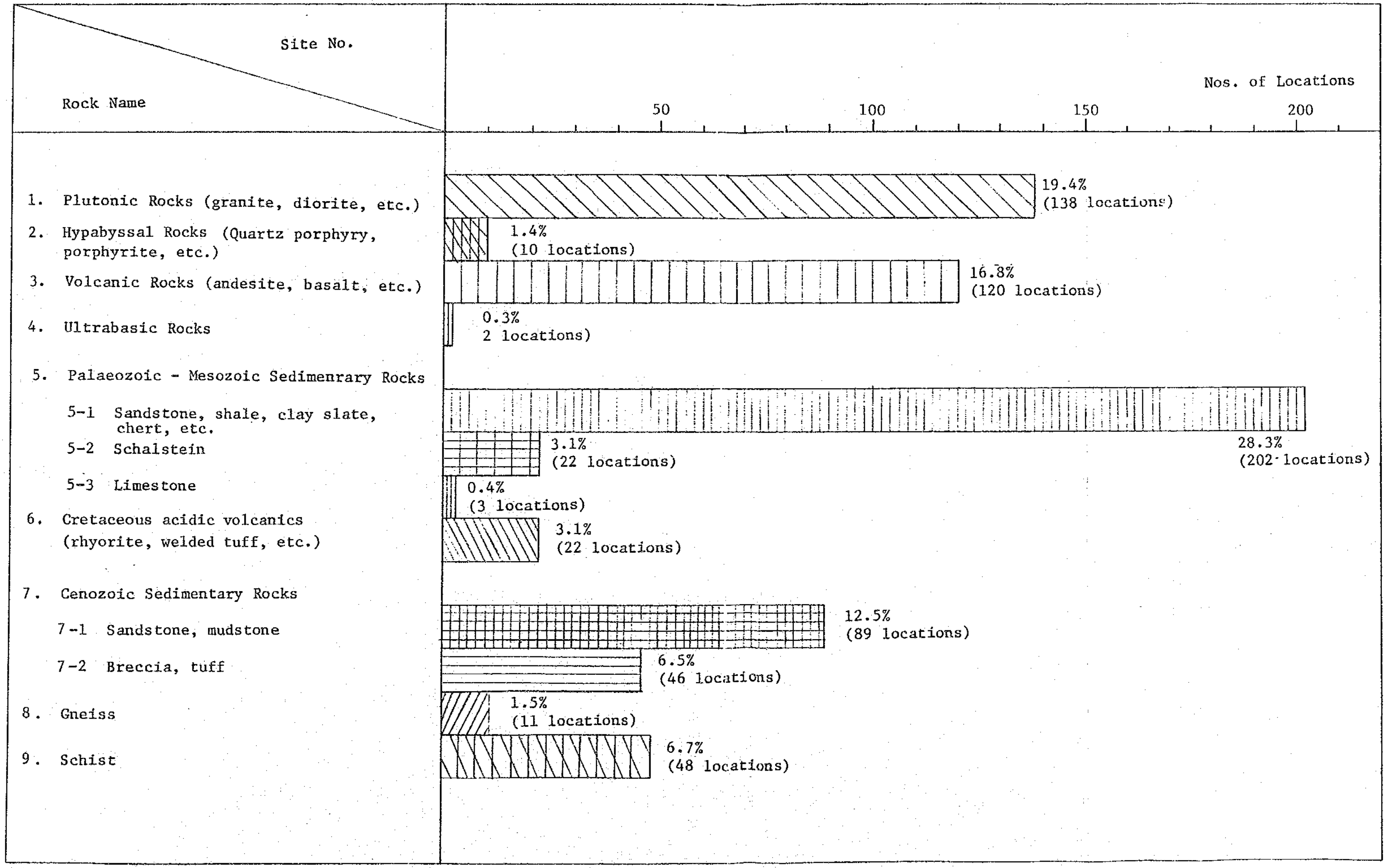


FIG. 2 Grauting Test Hole Arrangement

(at Cirata Site in Indonesia)

Primary groutig holes	:	6 m intervals
Secondary grouting holes	:	3 m intervals
Tertiary grouting holes	:	1.5 m intervals
Check hole	:	center

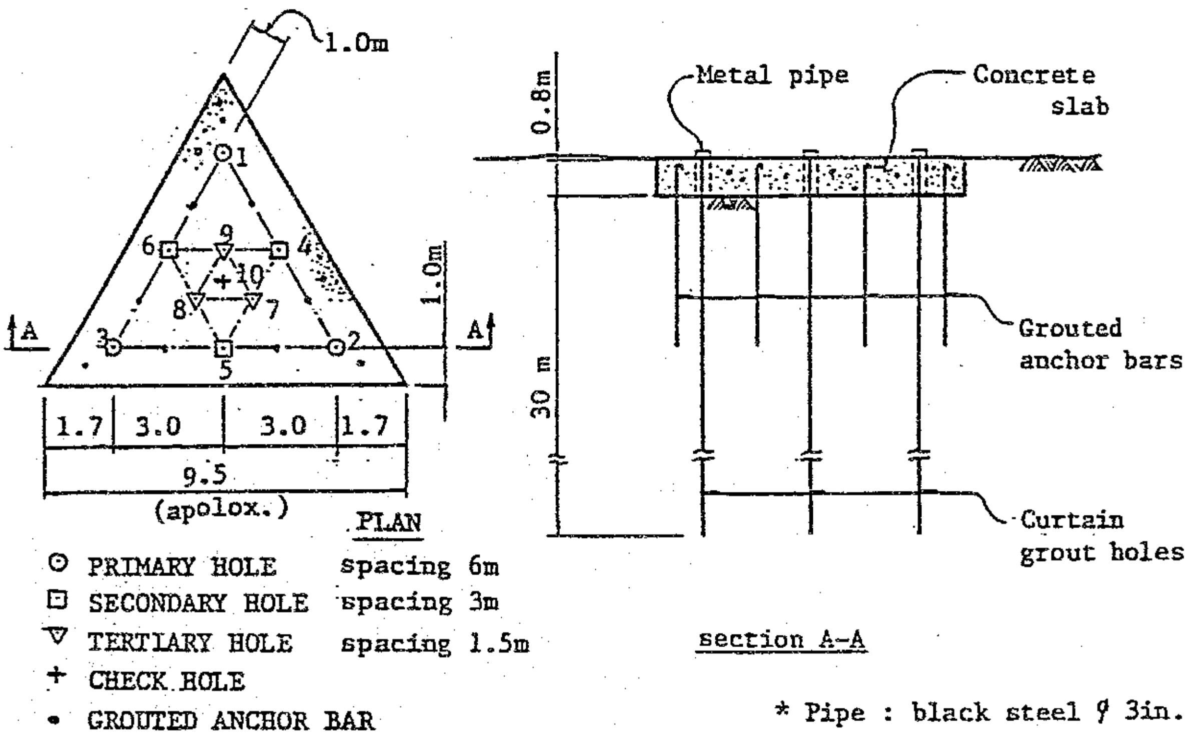


FIG. 3 Microseismic Station Network

(Saguling - Cirata Hydroelectric Power Projects)

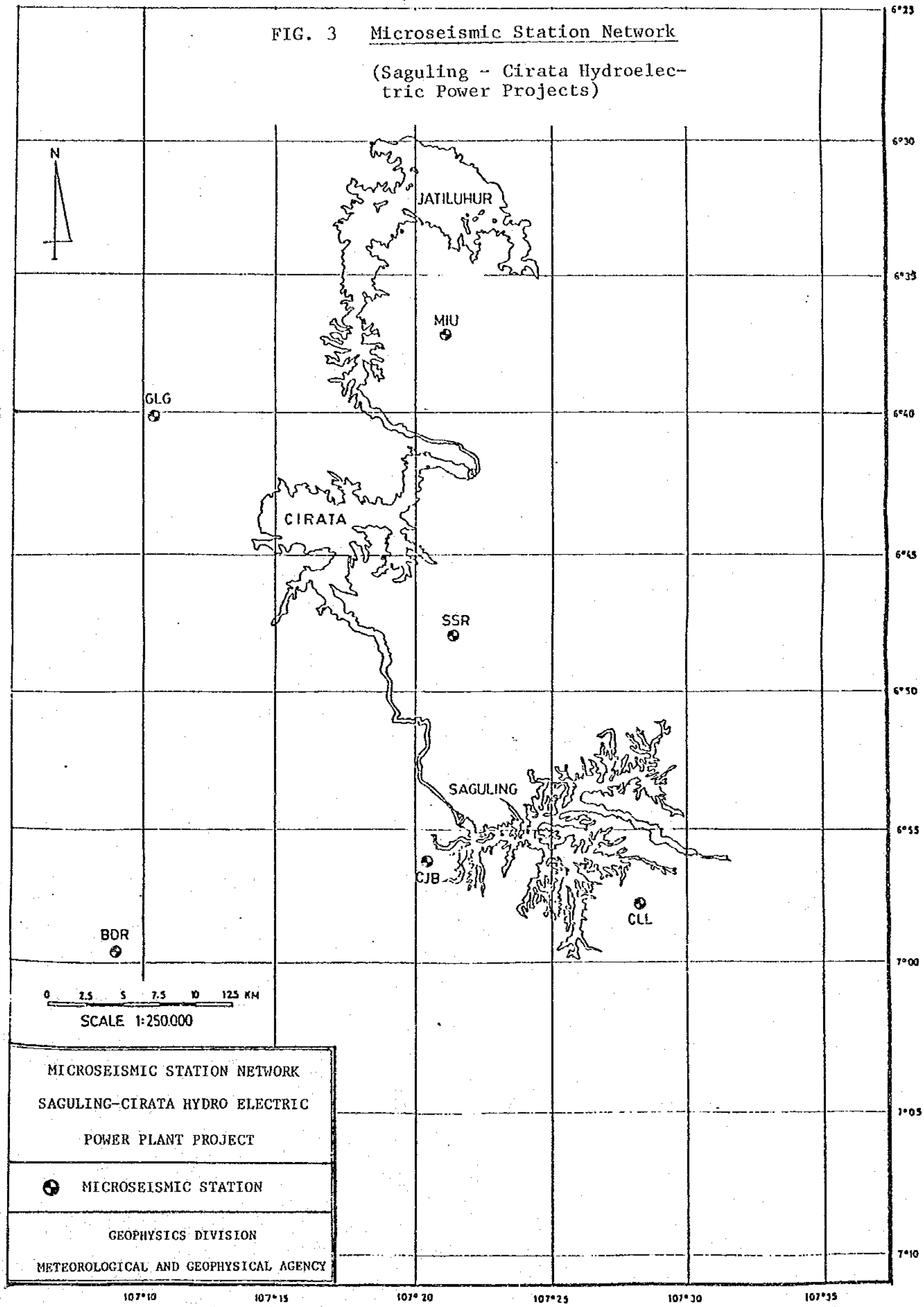


Table-1 CLASSIFICATION OF ROCK QUALITY

Class	Mode of Occurrence and Quality of Rock (By Dr. Tanaka)
A	Joints and cracks poorly developed, joint and crack planes are very tight, no weathering and/or alteration in rock mass. Cohesion of Joine and/or crack planes is very strong and there is no trace of weathering and/or alteration along these planes. The rocks metallic resound when struck by hammer, and belong to very fresh rock group.
B	Joints and cracks moderately developed, however, joint and crack planes are tight and cohesion of these planes is strong, no clayey or sandy materials in the joint and crack planes. Rock blocks separated by joint and crack planes are hard and compact, but rock forming minerals and/or grains along or the vicinity of these planes are sometimes slightly weathered or altered. (Some of them stained by limonite). The rocks metallic resound when strucl by hammer and belong to fresh rock group.
C C_H	Joints and cracks moderately developed. Generally, cohesion of joint and crack planes is moderately strong and are not easy to break by a strike of a hammer. However, some of them are opened slightly and filled with thin film of clayey and/or sandy materials. (Sometimes stained by limonite). Rock blocks separated by joints and cracks are moderately hard and compact, but rock forming menerals and/or grains are sometimes slightly weathered or altered. (Some of them are stained by limonite).

Class	Mode of Occurrence and Quality of Rock (By Dr. Tanaka)
C C_H	Rocks moderately metallic resound when struck by hammer and belong to moderately fresh rock group.
C C_M	Joints and cracks well deleloped. Cohesion of joint and crack planes is moderately weak, and some of openings of these planes filled with clayey and/or sandy materials. (Some of these materials are stained by limonite). The rock is broken along these planes by a strike of a hammer. Rock blocks or pieces separated by these planes are moderately weak and sometimes rock forming minerals and/or grains are weathered and/or altered. (Some of these materials are stained by limonite). The rocks moderately dull resound when struck by hammer and belong to moderately weathered and/or altered rock group.
C_L	Joints and cracks closely developed. Cohesion of joint and crack planes is weak and most of the opening of these planes filled with clayey and/or sandy materials. (Some of these materials are stained by limonite). The rocks are easily broken by a strike of a hammer along these planes. Rock blocks and pieces separated by joint and crack planes are moderate or weak owing to weathering and alteration. The rocks dull resound when struck by hammer and belong to weathered and/or altered rock group.

Class	Mode of Occurrence and Quality of Rock (By Dr. Tanaka)
D (1)	Rock forming minerals and/or grains are weathered and/or altered very much, and the rocks are easily broken by a light strike of a hammer. The rocks belong to very much weathered and/or altered rock group.
(2)	Large or moderately large open cracks and/or joints developed, most of them filled with clayey and sandy materials or vacant, and no cohesion among rock blocks and pieces, however, individual rock blocks or pieces are sometimes moderately fresh. The rocks belong to open cracked rock group.

Table-2 CLASSIFICATION OF ROCK QUALITY

(Saguling and Cirata Dam, Indonesia)

Class		Description	Remarks
B		No visible signs of weathering, Rock fresh, crystals bright. Few discontinuities may shown slight staining. Unweathered.	Fresh rock
C	C1	Penetrative weathering developed on open discontinuity surface but only slight weathering of rock material. Discontinuity are discoloured and discolouration can extend into rock up to a few mm from discontinuity surface. Slightly weathered.	
	C2	Slight discolouration extends through the greater part of the rock mass. The rock material is not friable. Discontinuities are stained and for contain a filling comprising altered materials. Onion structures are observed in such hard rocks as breccia and andesites. Moderately weathered.	Weathered rock
D	D1	Weathering extends throughout rock mass and the rock material is partly friable. Rock has no luster. All materials except quartz is discoloured. Rock can be excavated with geologist's pick. Highly weathered.	Badly weathered rock
	D2	Rock is totally discoloured and decompose and in a friable conditions with only fragments of rock texture and structure preserved. The external appearance is that of a soil. Completely weathered.	

TABLE-3 Concrete dams that have been subject to significant Seismic Shaking

Name, type, and year completed	Country	Height (ft)	Length (ft)	Date of earthquake (name)	Distance to fault	Earthquake magnitude	Measured horizontal acceler-ation	Remarks
Lower Crystal Spring, curved gravity - 1890	USA	154 (47m)	600 (183m)	April 18, 1906 (San Francisco)	0.25 miles (400m)	8.3	-	not the slightest crack
Hoover, curved gravity - 1936	USA	726 (22m)	1244 (379m)	Many* Starting in 1936	<5 miles (<8km)	5.0 max.		no damage
Honen-ike, multiple arch - 1930	Japan	97 (30m)	476 (145m)	Dec. 21, 1946 (Nankai)		VII-VIII intensity		crack in arch near buttress repaired by grouting
Blackbrook, gravity - 1900	England	100 (30.5m)	482 (147m)	Feb. 11, 1957	4 miles (6.4km)	Grade 8 on English Scale of 10		cracks in down-stream masonry face
Hsinfengkiang, buttress - 1959	China	344 (105m)	1444 (440m)	March 19, 1962*	3 miles to focus (5km)	6.1	0.54g in aftershock	craks in top part of dam
Monteynard, arch - 1962	France	509 (155m)	705 (215m)	April 23, 1963*	many shocks under dam	4.9 max.		no damage
Kariba, arch - 1959	Rhodesia	420 (128m)	2025 (617m)	Sept. 23, 1963* and others		6.1 max.		no damage
Kurobe, arch - 1960	Japan	612 (186m)	1558 (475m)	Many* 1963-1970	6 miles (<10km)	5.0 max.		no damage
Koyna, straight gravity - 1963	India	338 (103m)	2800 (835m)	Dec. 11, 1967* (koyna)	1.8 miles (3km)	6.5	0.63g along 0.49 trans-verse to axis	cracks in both faces
Grandval, multiple arch - 1959	France	289 (88m)	1312 (400m)	Many starting* in 1969		V intensity		no damage
Pacoima, arch - 1929	USA	372 (113m)	589 (180m)	Feb. 9, 1971 (San Fernando)	3 miles to fault break (5km)	6.6	1.25g	no cracks in arch-opening up of joint between arch and thrust block
Santa Anita, arch - 1927	USA	251 (76m)	670 (204m)	Feb. 9, 1971 (San Fernando)	17 miles (27km)	6.6	0.17g	no damage
Big Tujunga, arch - 1931	USA	251 (76m)	400 (122m)	Feb. 9, 1971 (San Fernando)	20 miles (32km)	6.6		no damage
Ambiesta, arch - 1956	Italy	194 (59m)	475 (145m)	May 6, 1976 (Gemona-Friuli)	14 miles (22km)	6.5	0.33g	no damage
Maina diSauris, arch - 1952	Italy	446 (136m)	453 (138m)	May 6, 1976 (Gemona-Friuli)	27 miles (43km)	6.5		no damage
Barcis, arch - 1955	Italy	164 (50m)	276 (84m)	May 6, 1976 (Gemona-Friuli)	30 miles (48km)	6.5		no damage
Poiana Usului, buttress - 1969	Romania	262 (80m)	1730 (527m)	March 4, 1977	37 miles (60km)	7.2		no damage

*Earthquake believed to be reservoir induced

(After K.D Hansen and L.H. Roehm 1979)

TABLE-5 Seismic Performance of Existing Rockfill Dams

DAM NAME (Location) [Year completed]	HEIGHT (ft)	SLOPE (hor/vert) (V/S)	SLOPE (hor/vert) (D/S)	EARTHQUAKE (name)	M	R (km)	PGA (g)	CA (g)	CREST MOVEMENT VERT. (cm)	CREST MOVEMENT HORIZ. (cm)	REFERENCE
MALPASSO (Peru) [1936]	255	0.5	1.33	10-10-1938	VI+ (1)	--	0.10 (2)	--	7.6	5.1	(1)
MORENA (Calif.) [1895]	177	0.5-0.9	1.3	04-06-1940	IV (1)	--	0.02 (2)	--	--	--	(1)
COGOTI (Chile) [1939]	275	1.6	1.8	07-04-1943 (Illapel)	8.3	16	0.20 (2)	--	38.1	--	(6),(41)
PINZANES (Chile) [1956]	220	1.2	1.3	28-07-1957	7.5	--	0.05 (2)	--	--	--	(1)
MIBORO (Japan) [1960]	430	2.5	1.75	19-08-1961 (Kitamino)	7.0	16	0.20 (2)	--	3.0	5.0	(31),(41)
MINASE (Japan) [1964]	220	1.35	2.0	16-06-1964 (Nigata)	7.5	16	0.08 (2)	--	6.1	4.0	(27)
LA CALERA (Mexico) [- -]	92	1.5	--	-- -- 1964	VIII (1)	--	--	--	--	--	(20)
KUZURYU (Japan) [- -]	419	2.6	1.8	09-09-1969 (Gifu)	6.6	40	0.02 (3)	0.04 (3)	--	--	(29)
KISENYAMA (Japan) [- -]	312	2.5	2.2	09-09-1969	6.8	--	--	0.10	--	--	(43)
OROVILLE (Calif.) (4) [1968]	770	2.75	2.0	01-08-1975 (Oroville)	5.7 (5)	6.9	0.10 (3)	0.12 (3)	0.9	--	(46)
TARUMIZU (Japan) [1976]	141	3.7	2.4	12-06-1978 (Miyagi)	7.4	100	0.24 (3)	0.36 (3)	--	--	(47)
EL INFIERNILLO (Mexico) [1964]	485	1.75	1.75	14-03-1979	7.6	110	0.12 (3)	0.35 (3)	13.0	4.5	(35)
LA VILLITA (Mexico) [1967]	197	2.5	2.5	14-03-1979	7.6	110	0.10 (2)	0.36 (3)	4.5	3.0	(35)
LEROY ANDERSON [1950]	235	2.0	2.0	24-04-1984 (Morgan Hill)	6.2	16	0.41 (3)	0.63 (3)	1.5	0.9	(44)

(After Gilles Bureau, Richard L. Volpe, Volfgang M. Roth, Takekazu Udaka, 1985)

(1) Modified Mercalli Intensity at Dam Site
(2) Estimated
(3) Recorded
(4) Earthfill dam with coarse pervious shell
(5) Magnitude 5.7 (Berkeley), 5.9 (USGS), 6.1 (Pasadena)

PGA = Peak Ground Acceleration (at or near base of dam)
CA = Peak Acceleration at Crest
M = Magnitude
R = Epicentral Distance

MAINTENANCE AND CONTROL

FOR

FILL DAM

July, 1986

Y. MATSUI
Civil Engineering Registered

JICA CALIRAYA DAM
REHABILITATION PROJECT

1. General

Dam control, prevailing in recent years, includes keeping security of dam structure and its foundation ground, area around dam reservoir, maintaining efficient functions of various facilities, and also environmental preservation.

This paper deals with maintenance and control aspects of fill-dam on the basis of existing fill-dam's performances, and refers to security of reservoir area.

2. Fill-dam Control

Main purpose of fill-dam control is to prevent such serious occurrences as overflow/topping of reservoir water at non-overflow dam crest, piping phenomenon and sliding of dam embankment and foundation ground. Those occurrences may cause dam destruction at high probability.

Ever since, lots of dams have been constructed and maintained, but some of the dams have experienced with grave accident. Most of modern technique in design, construction and maintenance and control of dams have been cultivated and developed by numerous efforts to remove and overcome the defects revealed in the past dam failures.

The general activities of dam structure maintenance/control necessary for securing dam safety consists of measurement, inspection, detail investigation and repair work.

A sequence among these are shown in Figure-1 as a flow chart.

If dam behavior is considered unusual in accordance with the results of measurement, inspection or detailed investigation, then necessary action such as appropriate repair or urgent countermeasure should be taken.

Judgement whether such unusual behavior indicates critical situation of safety or not and decision of what and when the countermeasure should be carried out are the most important actions of the maintenance/control activities, such judgement and decision should be done by a responsible engineer who is familiar with dam design, structural characteristics and control methods of the concerned dam.

2.1. Stage of Dam Control/Monitoring

Dam control period can be divided into several stages as described below:

a. Construction stage

The most important item of dam control during construction is pore pressure growth within dam embankment material. When high pore pressure growth is observed, slow down of embanking rate is to be considered. Measurement of settlement rate of dam embankment is also helpful for us to evaluate dam deformation in the future.

b. First stage

Period of initial reservoir impounding; behavior of a new dam under reservoir load is unknown until the dam is filled up to the full water level. Therefore, monitoring and inspection of a new dam at the time of initial reservoir impouding is most important. Most careful and close attention should be paid for dam control of this stage.

c. Second stage

Successive period of the first stage, until dam behavior comes to a steady status. Usually, the second stage continues for three years.

d. Third stage

After the second stage, until the expiry of dam service life.

Standard items of monitoring for the first to the third stages are listed in Table-1.

It should be noted that most of dam destruction and serious damage occurred in the past have taken place within a few years after completion of dam construction.

In general, dam behavior shows yearly and seasonal changes in accordance with atmospheric temperature and precipitation changes. But the rate or amplitude of changes becomes gradually smaller with elapse of time and approaches to an almost constant value.

Figures 2, 3, and 4 shows yearly changes of settlement, horizontal displacement and dam leakage at some rockfill-dams in Japan of more than 10 years old.

2.2. Monitoring of Dam

In case of an ordinary fill type dam, the fundamental monitoring items are two in zoned rockfill-dam and three in uniform embankment dam, i.e.:

- leakage and external displacement (settlement and deformation) for zoned rockfill dam.
- leakage, external displacement (settlement and deformation) and water table level in the dam embankment.

It is considered that the dam behavior is different from each other according to the topographical and geological condition of site, type and size of dams and construction method, therefore, besides items described in the above, some particular item of monitoring may be necessary for certain dams.

(1) Water leakage

Water leakage is to be monitored and controlled at any dam, because the leakages often link with surface instability and piping phenomenon which causes progressive erosions and slides at dam downstream slopes and even at foundation ground.

Items to be observed in connection with water leakage are the amount of water, location of leakage and muddiness of leaking water.

(a) Amount of water leakage

Upon evaluation of monitoring results, the following factors should be considered.

- water level of reservoir
  Generally, water leakage varies in accordance with the fluctuation of reservoir water level. Amount of leakage water is unstable at the initial impounding of reservoir; sometimes, leakage may suddenly increase at a certain reservoir water level after then, the amount of leakage will gradually approach to its steady value with time elapsing.

- permeability characteristics of dam body and foundation ground
  Amount of dam leakage consists of leakage through the dam body and from the foundation ground. In some cases, leakage amount from foundation may exceed that through dam body depending on the geological condition of dam foundation.

- influence by rainfall

  Usually, monitoring device (flow measuring weir) of dam leakage is installed at downstream area of dam embankment where surface flow of rainfall can easily join to the leakage from the dam. Thus, it is necessary to separate surface flow amount from that measured at the weir.

(b) Muddiness check

Leakage water from dam at normal condition does not include any suspended soil particles. Whenever soil particles are detected in the leakage water during inspection, the amount and size of particles should be carefully measured and cause of particle being washed out should be investigated. Such muddiness of leakage water is a significant sign of piping phenomenon.

Investigation of quality of leakage water often offers a useful measure to find out the location and route of leakage water.

(2) Deformation of dam body

Magnitude and distribution of deformation of dam body is one of important information in considering and evaluating dam safety, because the deformation observed at the surface of dam body are attributed to settlement of foundation, deformation of dam body created by external load and settlement of dam embankment.

Usually, measurement of settlement and displacement of dam crest and downstream slope surface are enough for monitoring purposes.

(3) Pore pressure/ground water table

Development of excessive pore pressure in the embankment material decreases shearing resistance of embankment and sometimes causes slope failures. Usually pore pressure incurred by dam embankment dissipates within a few years after stoppage of embanking work and reaches to a steady condition with normal pressure distribution corresponding to reservoir water.

It should be noted that even after the pore pressure reaches to a steady condition, there is a possibility of raising pore pressure in case of earthquake if embankment materials have high water content and low plastic index.

2.3. Site Inspection

Site inspection is indispensable for early finding of abnormality in dam and relevant structure/facility.

Inspection can be roughly divided into ordinary inspection, periodical inspection and special inspection to be carried out when dam areas are encountered with special phenomena such as heavy rainfall and strong earthquake.

Items to be inspected are variable according to type of dam and particularity of dam site.

Major items of site inspection for an ordinary dam are shown in Tables 2 and 3.

2.4. Investigation and Repair

When abnormal and damaged portion or unusual behavior are found out by monitoring and inspections, an adequate investigation should be undertaken to study its situation and to make the cause clear.

After investigation and study, if such abnormality is considered endangering the safety of structures/surrounding ground then an immediate repair work should be carried out. If determined as not emergent, but better to be watched, then further monitoring or investigation are to be continued.

Figure - 1

flow chart for dam control procedure

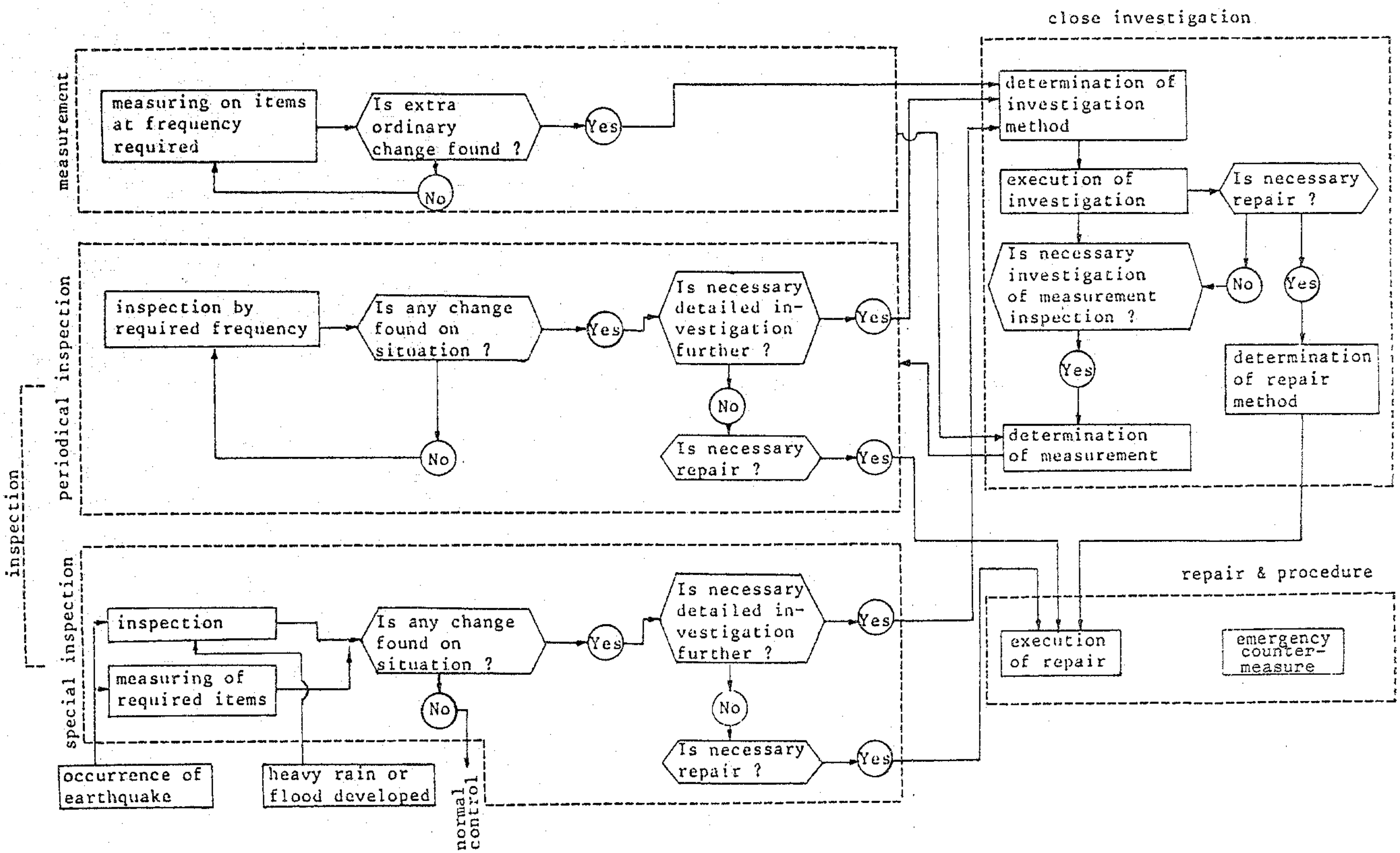
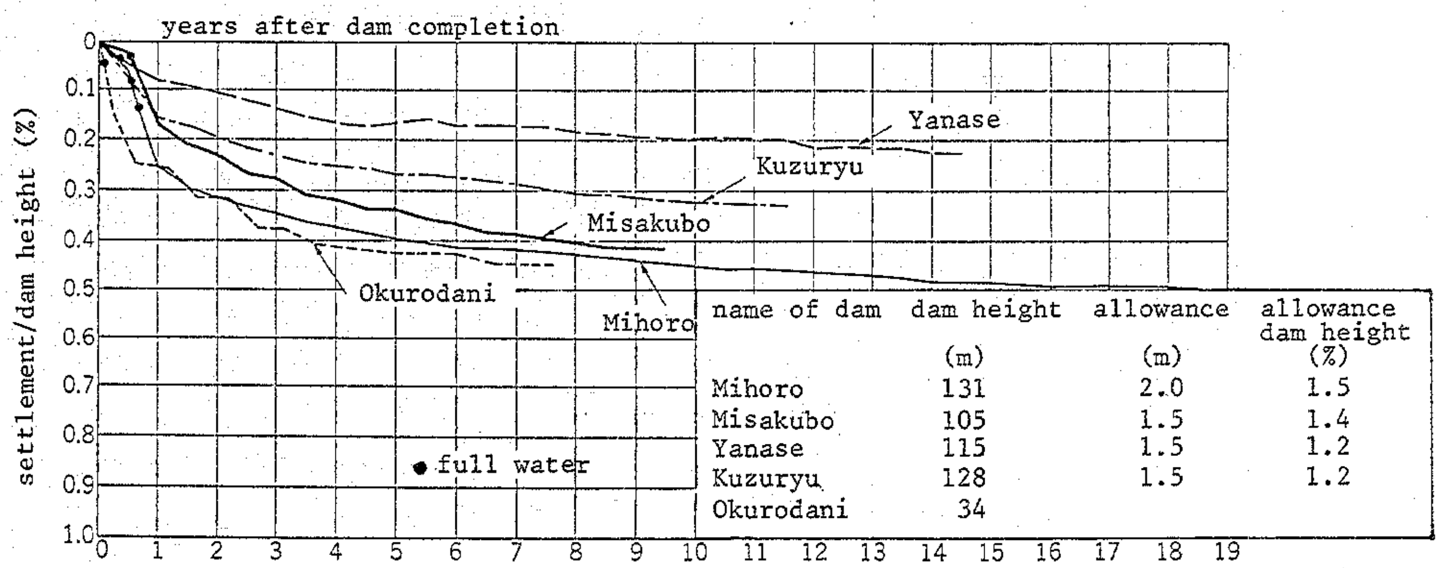
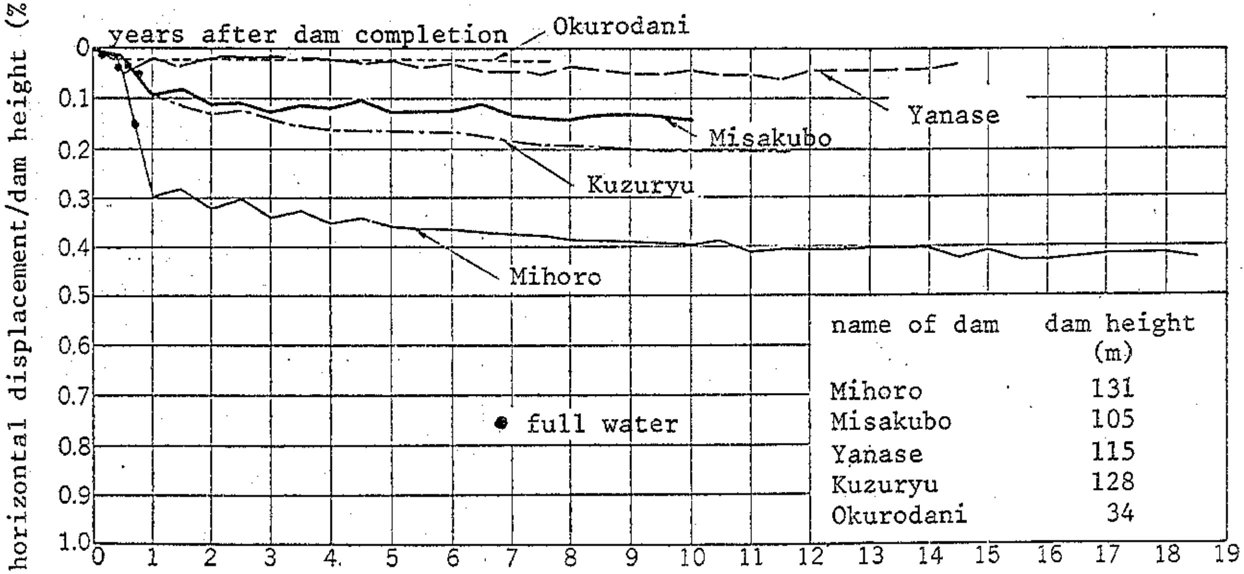


Figure-2 Yearly Change of Settlement at Dam Levee Crown



name of dam	dam height (m)	allowance (m)	allowance dam height (%)
Mihoro	131	2.0	1.5
Misakubo	105	1.5	1.4
Yanase	115	1.5	1.2
Kuzuryu	128	1.5	1.2
Okurodani	34		

Figure-3 yearly change of horizontal displacement at dam levee crown



name of dam	dam height (m)
Mihoro	131
Misakubo	105
Yanase	115
Kuzuryu	128
Okurodani	34

Figure-4 Yearly Change of Water Leakage

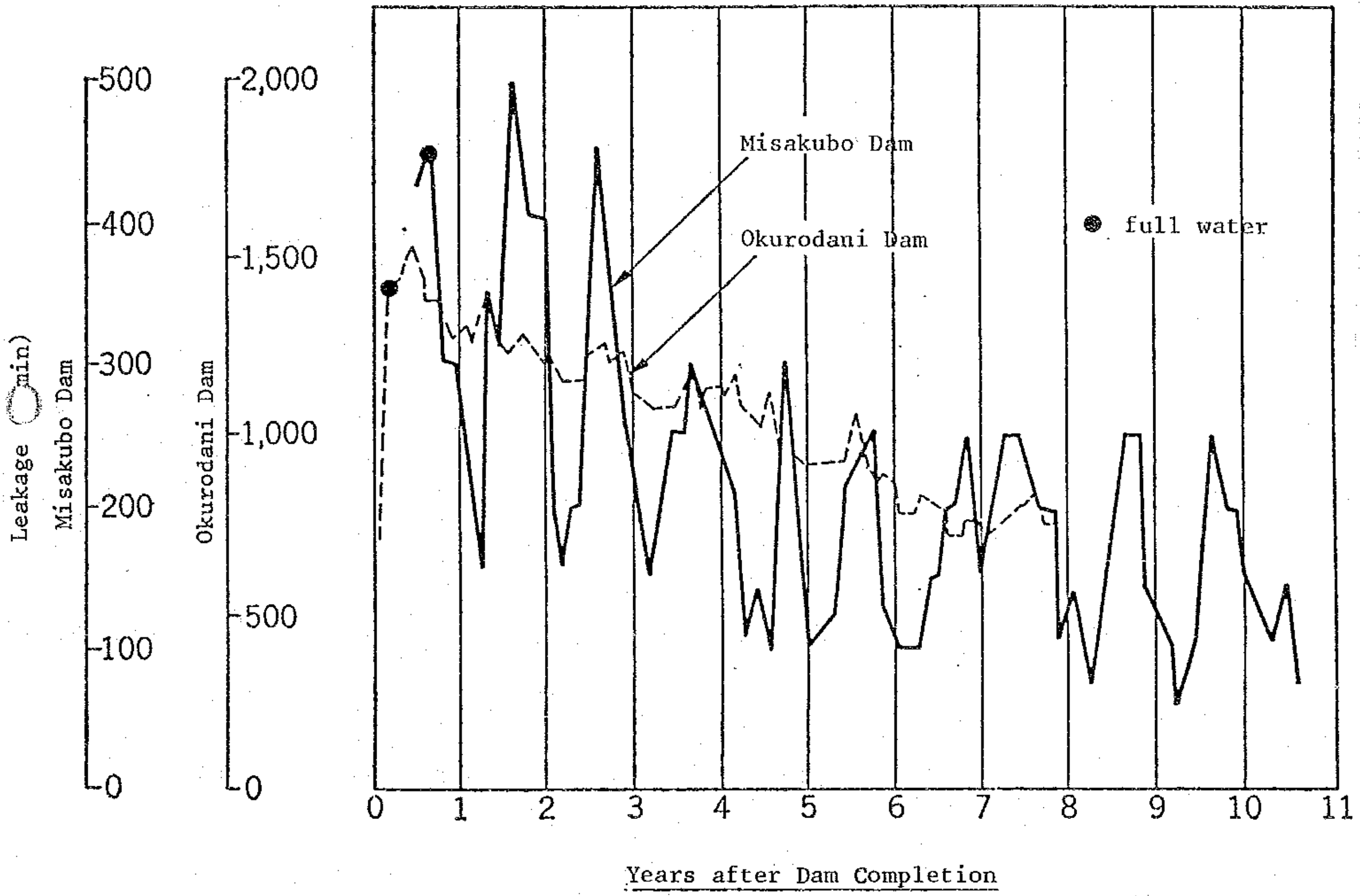
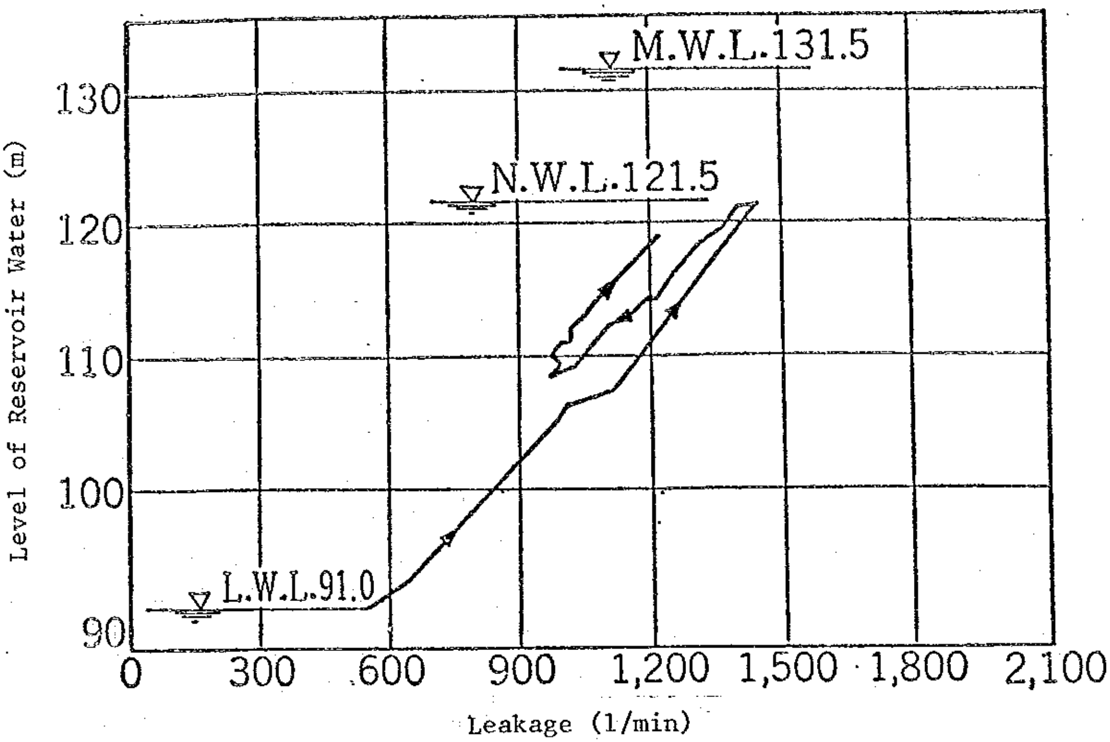
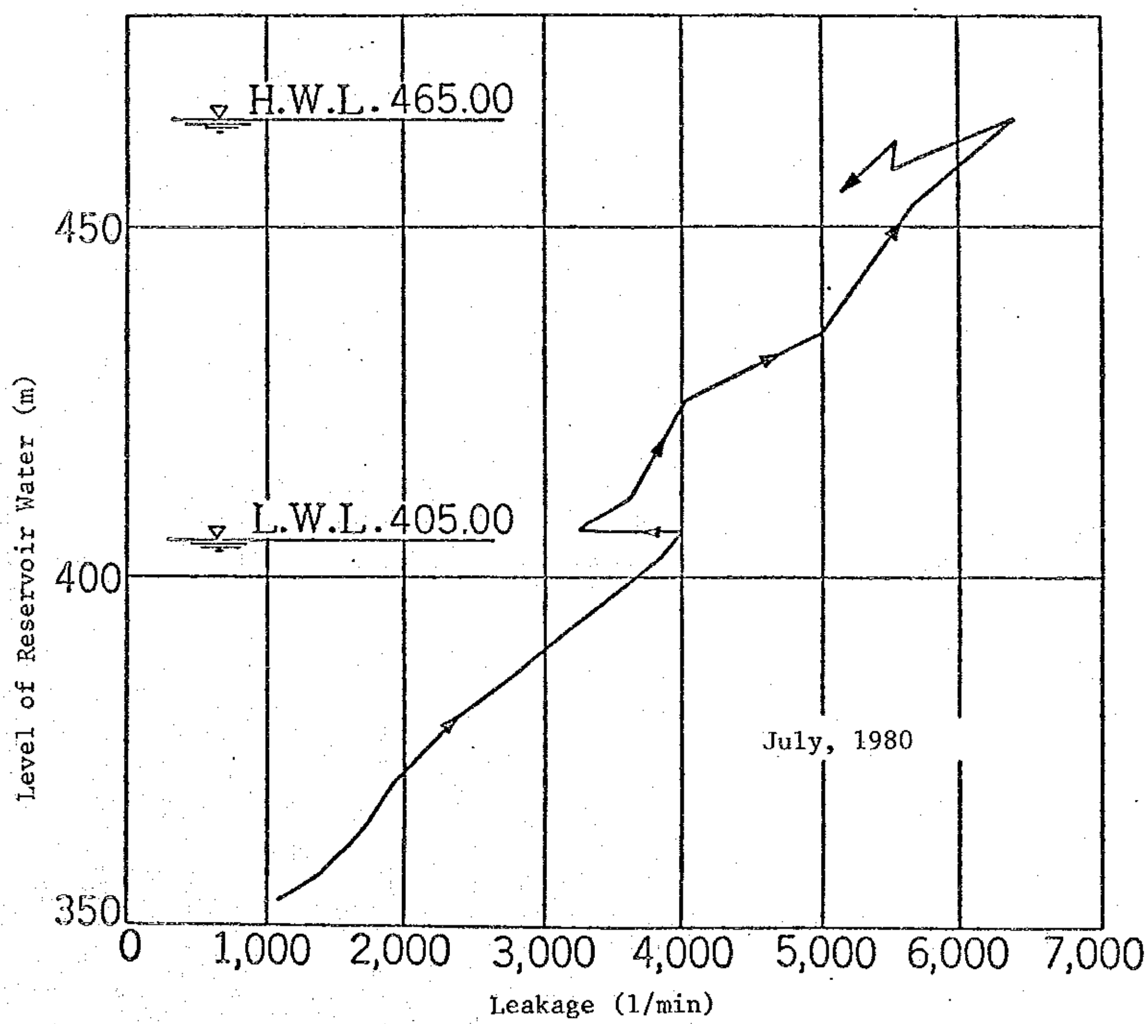


Figure-5 Relation between Reservoir Water Level and Leakage



(a) Terauchi Dam



(b) Tedorigawa Dam

Figure-6 Yearly Change of Water Seeping at Asphalt Impervious Wall (Otsuki Dam)

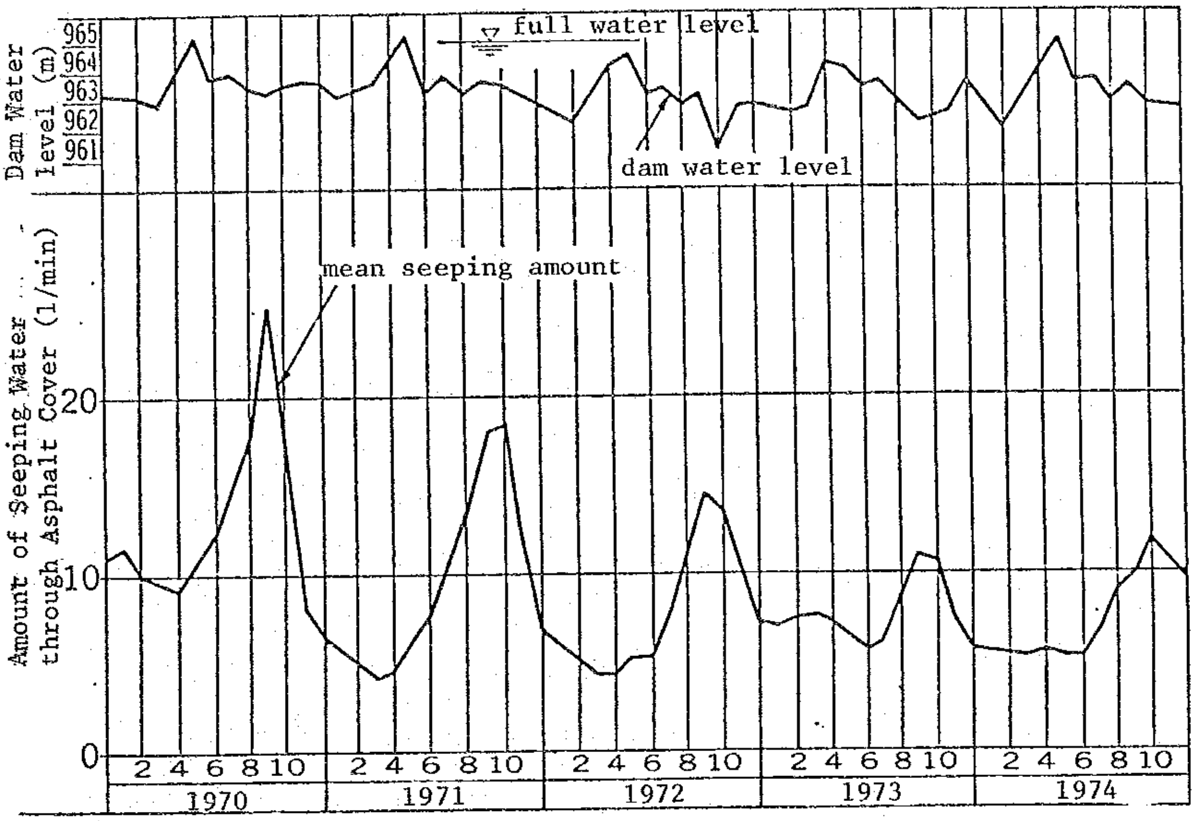
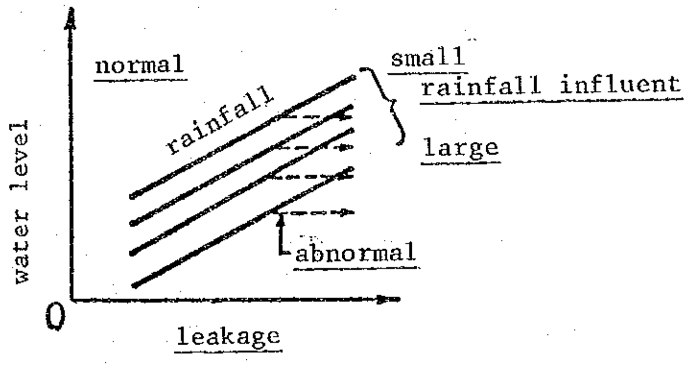
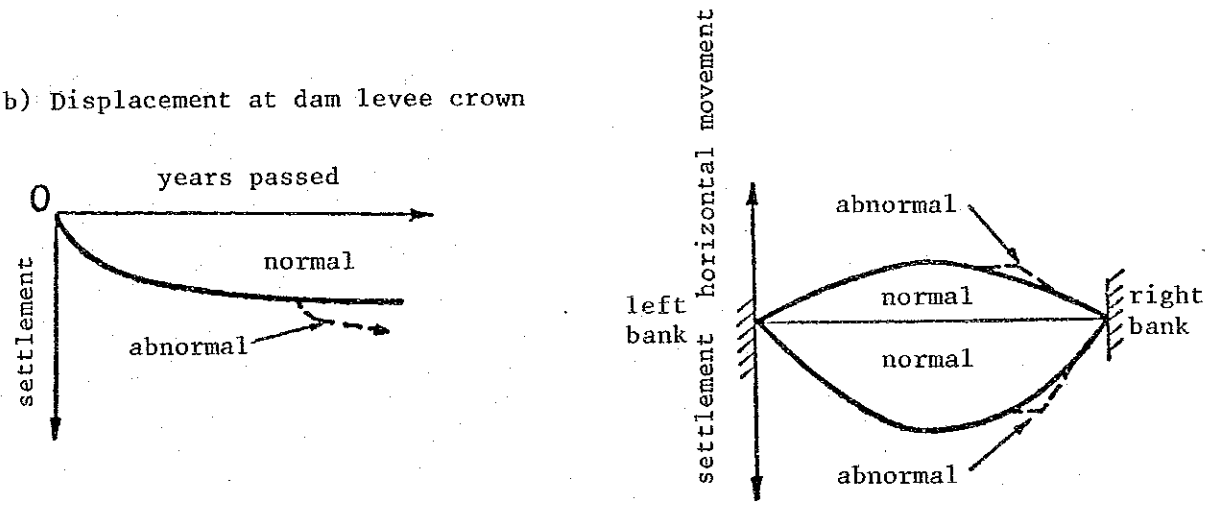


Figure-7 Stability Judgement of Fill Dam

(a) Leakage



(b) Displacement at dam levee crown



(c) Pore pressure

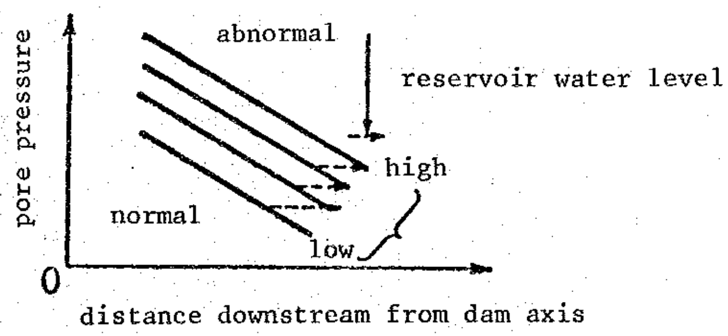
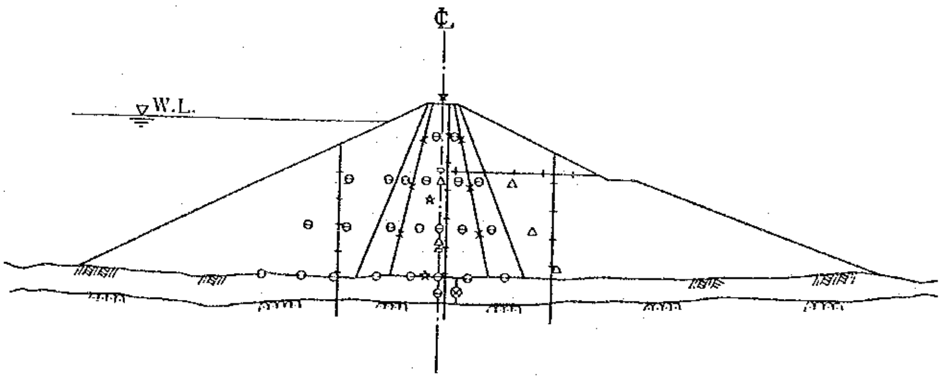


Figure - 8 Example of Instrumental Arrangement



	legend
⊖	pore pressure meter
△	earth pressure meter
⫲	settlement at separate layer (inclinometer)
-+-+-+-	horizontal displacement meter
X	slide displacement meter
□	soil strain meter
✪	seismograph installation
⊗	rock displacement meter

Table - 1 standard list of measurement items and frequency

stages	dam type & height	items	leakage	deformation	seepage line
first stage	surface impervious wall type	-	everyday	once a week	
	zone type	-	everyday	once a week	
	uniform type	-	everyday	once a week	once a week
second stage	surface impervious wall type	-	once a week	once a month	
	zone type	-	once a week	once a month	
	uniform type	-	once a week	once a month	once a month
third stage	surface impervious wall type	less than 70m more than 70m	once a month once a month	(once three months) (once three months)	
	zone type	less than 70m more than 70m	once a month once a month	(once three months) (once three months)	
	uniform type	-	once a month	(once three months)	(once three months)

Table - 2 standard of special inspection

object	target size decided before hand	quality of inspection
earthquake	1/3 of design magnitude of earthquake (in case of seismograph installed in dam) IV degree by Meteorological Agency scale (in case of no seismograph in dam)	measurement of required items and inspection same as periodical inspection
flood	flood occurring once in 3 years	inspection same as periodical inspection
heavy rain	daily rainfall developing once in 3 years	

Table - 3 inspection items & inspection particulars

points	items	particulars
dam levee crown	crack	developed or not (direction, width, depth etc.)
	settlement	disorder like depression
	deformation	disorder like curve or bend
	security fence	any defection
slope	protection work	damage to riprap, impervious wall
	erosion	situation of erosion and damage to slope face
	settlement	depression, opening, crack
	deformation	disorder like swell
	vegetation	vegetation situation
	water spring	seepage, saturation point
surrounding bed rock	slope	slide, collapse
	crack joint layer	any disorder
	water spring	new spring, disorder of existing springs (amount, water pressure, muddiness)
	snow avalanche	situation
measurement locations	instruments	operational situation
	record	normal or abnormal

Table - 4 Comparison of Instrument Type

Type / Items	Carlson	Potentio meter	Strain gauge	Differencial Trans	Vibration Wire	Grötzel	Remark
characteristics	not affected by temperature change	excellent on detect displacement large	very high resistant to shock force	almost no energy needed for conversion (no need of mechanical power for detection of displacement)	applicable at broad temperature range due to independent from electrical resistance	no electrical system and no cost of transmitting	◎ best ○ good ✕ not suitable
displacement meter	○	◎	○	○	✕	✕	to 50mm
displacement meter	✕	◎	○	○	✕	✕	50∿1,000mm
tilt meter	○	✕	○	◎	◎	✕	20 sec/reading
earth pressure meter	◎	✕	◎	○	◎	◎	
water pressure meter	✕	✕	○	◎	✕	○	less than $1kg/cm^2$
water pressure meter	◎	✕	◎	○	◎	◎	more than $1kg/cm^2$

Table - 5 List of Dam Monitoring Facilities in Japan (Examples)

A : Numbers installed
B : Numbers troubled
( ): Numbers of component

item	instrument	type	Mihoro		Makio		Honzawa		Yanase		Kisen-yama		Shimo-kotori		Misakubo		Kuro-kawa		Nii-kkapu		Tatara-gi		Abura-tani		Uchi-tani	
			A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
pore pressure	pore pressure meter	Piezometer	52						23						35											
		Carlson					44		3														40	5	48	6
		Differential Trans									31	8	24	5			10	3	32	6	4	0				
		Ballanced Valve									17	7														
		Strain gauge																								
stress	earth pressure meter	Carlson					10																7	0	6	1
		Differential Trans									24	10	22	5			4 (12)	5	4	2	4	0				
		Ballanced Valve									39	22														
internal deformation (settlement)	layer settlement	Cross arm																					1	1	1	0
		Magnetic					1				8	0	4	0			2	1	1	0			2	2	2	1
internal deformation	relative displacement meter												7	0			3	0								
	shear displacement meter																		1	0			4	2	2	1
	slide displacement meter																									
	horizontal displacement meter																									
	horizontal-vertical displacement meter												5	0							16	0				
	soil strain meter																		4	2						
	rock displacement meter																									
earthquake movement	seismograph	Displacement meter	6		3						8	0	1	0							3	0				
		Acceleration type	11		16		8		9		32	1	11	0			4 (10)	0	14	0	16	0	4 (12)	0		
		Smack type													12											
leakage	triangle, rectangle weir, etc.		1				1		1																	
	automatic recording water level meter										3	0	3	0	1		1	0	1	0	1	0	1	0	1	0
external deformation	slope face displacement meter												1	0												
	crest displacement meter												3	0												
	survey target, pile		34	0	20		28		28		33	0	46	0	12		40	0	17	0	42	0	22	0	30	0

INVESTMENT PLANNING

FOR

HYDRO POWER FACILITIES

July, 1986

HAYAO ADACHI
Hydro Power Planning Engineer
(Civil Engineer)

JICA CALIRAYA DAM
REHABILITATION PROJECT

INVESTMENT PLANNING FOR HYDRO POWER FACILITIES

H. Adachi

1. Basement of Economics

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   1.2. Economic Evaluation/Financial Evaluation
   1.3. Interest Rate/Fair Return/Opportunity Cost
   1.4. Comparison of Alternatives
   1.5. Benefit Maximum/Ratio Maximum

2. Instruments for Power Planning

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   2.3. Geology
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   2.5. Construction Cost

3. Optimization of Project/Structure Components

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   3.3. Plant Factor
   3.4. Reservoir Dimensions
   3.5. Optimization of Structure Components

4. Conception of IRR for Rehabilitation Investment

INVESTMENT PLANNING FOR HYDRO POWER FACILITIES

1. Basement of Economics

1.1. General

Hydro power plannings are unable to be performed without primary knowledge of economics of investment. A power project is generally planned, studied and implemented by a team which comprises experts from several fields such as civil engineers, power engineers, geologists, economists, ecologists, system engineers, electrical engineers, mechanical engineers and so on. However, the person who manage the team should have basic knowledge of the economic evaluation for investment.

1.2. Economic Evaluation/Financial Evaluation

A feasibility study of a project generally requires two kinds of evaluations; economic evaluation and financial evaluation. The understanding for the difference of real meanings between the two may afford a principle recognition of a project evaluation. An evaluation is unable to be carried out with an absolute analysis of the project. The evaluation should be always executed by applying a comparative analysis, which requires alternative projects for the comparison. The selection of alternatives for the study depends on a system to be considered.

(1) Spectrum of system

An evaluation of a project can be conducted with either world-wide/nation-wide spectrum or an financial unit/project-wide spectrum. It depends on a person who requires the evaluation. Even though some project may not require the study of the world-wide or nation-wide, generally speaking, such projects as hydro powers should have, at least, enough wider spectrum for their evaluation, because the investment for a hydro power development should be made for welfare of human-beings.

A project is unable to be realized only with a wider spectrum, because it is impossible to find the person who provides finance to a project without study of finance-unit-wide survey. Also, it is essential to evaluate a project with a narrower spectrum to satisfy a financial agency or a person who manage a hydro power plant through its life time.

(2) Difference of methods between two, wider/narrower

For instance, in case of the wider-spectrum, the fuel cost of a thermal power plant as an alternative for hydro power plant used to be applied referring to an international price/a shadow price. If the economy is clarified by applying the international price/the shadow price or secondary benefits, it means that the project is economical from the view-point of the world-wide economy.

Even though the project is clarified to be economical with the above method, it is not always for the project to be realized, because the person or agency who shall develop the project is unable to obtain return/benefit as computed by applying the wider evaluation method.

(3) Policy/strategy of development

As mentioned, it is essential for a project to be economical at least in the spectrum of the world-wide/nation-wide. Once it is clarified to be economical in the wider spectrum, several strategies can be created to realize the project implementation.

Generally speaking, secondary benefits are difficult to provide actual finance without any strategies. However, it is possible to discuss how to obtain the finance from other sectors analyzing the results of the economic study with the wider-spectrum.

(4) Economic/financial evaluation

Summarizing the above discussion, it is stated that the economic evaluation clarifies the project to be economical from the viewpoint of the world-wide/nation-wide and the financial evaluation explains how to provide finances to the project.

1.3. Interest Rate/Fair Return/Opportunity Cost

The evaluation of investments is to be carried out applying time-series analysis, which requires complete understanding of meanings for the conception of interest rate/fair return/opportunity cost. Generally speaking, Internal Rate of Return (IRR) is applied for the evaluation of the project. The IRR computed should be evaluated in the light of economic status in the system. It is not merely an interest rate in banks, but also comprehensively integrated fair return in the system.

1.4. Comparison of Alternatives

When computing the IRR, economic data of alternatives to be compared with the project are to be provided. The selection of alternatives is essential for the project evaluation. The definition of "value" is recognized as the least cost among those of alternatives.

1.5. Benefit Maximum/Ratio Maximum

The project is ultimately evaluated by computing IRR. However, in the course of studies, several optimizations of project dimensions or structure components are to be conducted applying the conception of benefit net value or benefit/cost ratio under the assumption of a certain value of the fair return.

The optimal point is not always same between points obtained by benefit-maximum (V-C) and ratio maximum (V/C) methods.

It is recognized that the ratio maximum (V/C) method is to be applied to the selection of sites or time-sequence of the development and the benefit maximum (V-C) method to the optimization of the certain project or of the structure components. This difference came from the fact that a system economy can be realized by adopting project materials in the order of larger value of V/C in the time series and, on the other hand, the projects/structure components which have been implemented are unable to be alternated in other opportunities.

2. Instruments for Power Planning

2.1. General

The power planning requires several technical instruments not only of basic engineering knowledges but also of hydrology, river water management, geology, system analysis, cost evaluation and so on. The power planning work should establish the project layout plan mobilizing several instruments with complete understanding.

2.2. Hydrology/Watermanagement

The most basic data to be necessary for a hydro power planning is river run-off data throughout past several years. It is usual to be difficult to find complete data of river discharge through years. In this case, several hydrologic technic is to be applied to synthesize river discharge data on the basis of rain-fall records or others. The economic evaluation of the project is to be carried out by understanding the procedure of the estimation for the basic data of river discharge.

The computation of water management through a river provides estimated generating energy which is to be basis of economic/financial evaluation. At the execution of the water management, the following points are to be attentioned:

(1) A necessary number of years for the water management (reservoir operation simulation) should be designated in the light of availability of discharge-data, reliability of the data, conception of precipitation cycles and so on.

(2) Boundary conditions are to be designated prior to the decision of cases to be examined. They are, for instance, necessary discharges to be provided to other sectors such

as irrigation and municipal/industry water, reservoir operation rules of existing reservoirs and so on.

(3) Integral operation through a river is usually to be considered. Even though the operation rules of existing reservoirs have been decided, it may be necessary to examine whether it is possible or not to re-optimize the rules in the light of the implementation of the new reservoir. Target function of the optimization is also essential for the decisions of the simulation method.

(4) Selection of cases to be studied depend on the alternatives of reservoir dimensions to be examined and, also, on the alternatives of plant capacities.

2.3. Geology

The hydro power planning engineer should have, at least, basic knowledge of geology, especially for the study at the inception stage of the project development. In the preliminary stages, the planning engineer is required to make a decision with the least data of geology.

The understanding of geologic history used to help the decision of the layout for the structure components with the least data of geology. As it used to be suggested by geologists, even in the stage of preliminary without any geologic investigation works, mountains provide several geologic informations and these informations are unable to be obtained without reconnaissance by the project planning engineer by his own even accompanied by geologists.

As it is anticipated, once a finance is provided, a large scale of investigation works are able to be conducted and a small scale of the investigation works conducted in the earlier stages seems to be meaningless. However, it is not true, because the decision at the earlier stage is extremely essential to provide a finance further to proceed the project with the least data of geology.

2.4. System Analysis

The power system analysis is essential for the power project planning. The economy of a hydro power plant is unable to be evaluated without examining a total power system and several simulation programs have been developed for this purpose. Optimization of small parts of the project is able to be studied without the optimization in the power system. However, at least, the following items are unable to be optimized without consideration of a total power system;

(a) Timing of the implementation of a project
(b) Decision of a plant factor of a project
(c) Sometimes, reservoir dimensions

The power system simulation is generally executed through future 10 or 15 year-span to obtain a total present worth of the system cost. The system costs are to be compared among several alternative factors of the project such as a plant factor, timing of implementation and so on.

Also, the simulation provides a value of benefit for the computation of Internal Rate of Return.

2.5. Construction Cost Valuation

It is natural that the hydro power planning is unable to be studied without estimation of construction costs. However, the actual construction method depends on several aspects of a project and the construction costs of many cases should be computed at the stage of the optimization. The following should be taken care at the computation of the construction costs;

(1) Usual efforts of accumulation of actual examples contribute to the accuracy of the estimation.

(2) Theoretical variation of the costs due to the variation of several dimensions of the alternatives should be able to be computed.

(3) The basis of the costs for labourers, materials and equipment should be clear in the cost organization because of necessity to review the costs in the light of current economic status.

3. Optimization of Projects/Structure Components

3.1. General

Before comparison among alternative projects, a hydro power project should be optimized. Precisely saying, these optimization works should be carried out in relation with other projects. However, almost of items are independently optimized under the assumption of unit-benefits and an interest rate. The optimizations of implementation timing and of a plant factor are unable to be made without consideration of a total power system.

3.2. Implementation Timing

The examination of the implementation timing of a project is to be conducted in the total power system simulation obtaining a total present worth of the system cost through out years. The important factors for the power system simulation are the following;

(1) accuracy and range of the estimation of power demand growth

(2) exact estimation of base load powers

(3) careful selection of combination among alternatives in the light of actual status.

3.3. Plant Factor

Even though the dimensions of the reservoir and output from the reservoir have been fixed, the maximum capacity of a hydro power plant is unable to be optimized until examination is made in the power system simulation, because the plant factor has close relationship with the organization of supply powers in the system.

The plant factor of a hydro power plant generally depends on the following;

(1) a load factor of the system
(2) alternatives of peaking supply power
(3) economic characteristics of the project

The conventional method of the optimization for the plant factor of a hydro power plant used to give a low plant factor as optimal. However, the low plant factor often used to cause shortage of energy in the system operation. When a low plant factor is going to be designated, precise studies for other peak supply power such as gas turbines and pumped storages.

3.4. Reservoir Dimensions

Main factors of a reservoir planning are the following;

(1) location of dam site
(2) maximum water level
(3) reservoir depth for utilization

An optimal combination among the factors is to be examined by applying a feed-back conception.

(1) Location of dam site

A dam site is generally decided from the view-point of dam construction engineering rather than economic view point because of essence of safety. However, in case of that several alternatives of the dam locations can be proposed from the dam engineering view-point, economical optimization is important computing benefits for several dam sites. The several location of dam sites may afford several values of possible maximum water levels and, sometimes, of drainage areas which contribute to the project economy. The bases of the comparison among several dam sites are, of course, the estimation of construction costs to be obtained on the basis of dam construction engineering aspects such as geology, dam type selection and so on.

(2) Optimum maximum water level

The maximum water level or the dam height at the certain dam site is to be optimized computing benefits based on head difference for generation and a firm discharge to be obtained from a reservoir capacity. The optimization of this maximum water level also depends on the construction cost estimation on the basis of the dam construction engineering.

In some cases, a dam height is unable to be optimized without consideration of integral development through a river. The conception of multi-stage development of a river is sometimes essential as an alternative of the optimal maximum water level of the reservoir.

(3) Reservoir depth for utilization

Under the assumption that the maximum water level of the reservoir is fixed, the depth of utilization for power generation is able to be varied. Therefore, the depth should be also optimized examining outputs for several alternatives of the depth.

Generally speaking, when increasing the depth, the firm discharge will increase due to larger capacity of the reservoir and, on the other hand, an average head difference will decrease.

In case of that a main portion of the head difference for generation is obtained by dam height, a shallower depth may accompany more benefit than that of the case that the head difference is obtained by utilizing an actual topography.

3.5. Optimization of Structure Components

Almost of the components are able to be optimized by applying the maximum net benefit method under the assumption that unit benefits and an interest rate are constant, respectively. It is of course that a sensitivity analysis should be conducted for the economic factors, if necessary.

The factors to be initially or principally optimized for the purpose of layout design are the following;

(1) water way system
(2) dam type
(3) flood routine
(4) river diversion system
(5) power house type

4. Conception of IRR for Rehabilitation Investment

In case of a new plant construction, the IRR is applied as an index of a method of investment which has been required in relation with system demand and supply. On the other hand, in case of the rehabilitation, the IRR is applied as an index for the decision of economic necessity of investment or optimal implementation timing.

Accordingly, the delay of investment for rehabilitation may not cause serious unbalance in demand-supply relationship and only may cause economic demerit due to the delay of recovering output an efficiency of generation. Therefore, even the investment is postponed in this year, the opportunity of the investment must again come in next year.

On the basis of above discussion, it is concluded that the IRR for the decision of rehabilitation investment is to be computed by comparing net benefit between the cases of investment in this year and in next year.

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