EARTH AND ROCKFILL DAMS

BARRAGES EN TERRE ET EN ENROCHEMENT

Chairman/Président: A. MAYER (France); General Reporter/Rapporteur Général: S. D. WILSON (U. S. A.)
Associate Reporter/Rapporteur Adjoint: R. SQUIER (U. S. A.)

Members of the Panel/Membres du Groupe de Discussion:
A. CASAGRANDE (U. S. A.)
R. J. MARSAL (México)
A. NITCHIPOROVITCH (U. S. S. R.)
I. L. PINKERTON (Australia)
P. SEMBENELLI (Italy)

Président A. MAYER (France)

Messieurs, mes premiers mots seront pour remercier le Comité Organisateur du Congrès, qui m'a fait l'honneur de me confier la Présidence de cette réunion. Je remercie également toutes les personnalités éminentes qui ont accepté les tâches plus actives que la mienne, de Rapporteur Général, et de Membres du Bureau. Avant de donner la parole à notre Rapporteur Général, je voudrais indiquer certaines dispositions que nous comptons adopter aujourd'hui:

1o Tout d'abord il n'y aura qu'une interruption de séance au lieu des deux initiallement prévues. Nous la ferons après le rapport général et les interventions des Membres du Bureau et avant de passer à la discussion.

2o Il nous est apparu que les communications préparées à l'avance alors qu'une minorité d'auditeurs seulement avaient reçu les rapports du Congrès ne pouvait faire l'objet d'une discussion mais seulement d'un complément d'information et qu'elles pourraient aussi bien être présentées par écrit et insérées dans le volume à paraître des compte-rendus du Congrès.

Nous avons donc décidé qu'il n'y aurait aucune intervention du type habituel. Je souligne, aucune. Tous ceux parmi les participants qui étaient inscrits et qui avaient préparé des textes pour les présenter à la deuxième partie de la séance, sont invités à les remettre en vue de leur publication.

Nous leur demandons toutefois bien entendu de se munir du papier spécial pour que cette publication ait lieu en Offset conformément aux règles édictées pour les premiers rapports. Par contre, tous les auditeurs sont invités à nous soumettre quand ils voudront, des maintenant, ou au cours de l'interruption, les textes des questions qu'ils désireraient voir discuter ou sur lesquelles ils voudraient avoir l'avis du bureau et éventuellement de l'assistance. Ils voudront bien remettre ces questions au Secrétaire de séance Monsieur Moreno, qui est ici, soit au cours de la séance, soit pendant l'interruption.

Nous nous efforcerons de les classer, de poser ces questions et de voir qui pourrait y répondre de la façon la plus satisfaisante.

Non seulement d'ailleurs ces réponses pourront être données par une personne du Bureau, mais encore si des personnes de l'assistance avaient des compléments à ajouter, nous essayerons de leur donner la parole bien que le nombre des assistants ici rende difficile une organisation de cette nature. Nous espérons que les modalités que nous vous proposons donneront plus d'animation à cette séance et dans cet espoir je donne tout de suite la parole à notre Rapporteur Général. Vous connaissez tous Monsieur Stanley Wilson, d'après le rapport magistral qu'il a présenté.

D'autre part, tous les ingénieurs mexicains savent que depuis 10 ans il est conseil de la Commission Fédérale d'Electricité et du Secrétariat des Ressources Hydrauliques où il a participé à l'établissement des projets de tous les grands barrages en enrochement réalisés ou en cours d'éxécution au Mexique, au cours des dernières années.

Si je vous indique que Monsieur Wilson est Ingénieur Conseil, qu'il est un des membres du Bureau Shannon et Wilson de Seattle aux Etats-Unis, qu'il a étudié à Harvard sous la direction du Professeur Casagrande. Je crois que vous en savez essez et que son rapport général vous dira le reste. Je passe la parole à Monsieur S. Wilson.

General Reporter S. D. WILSON (U. S. A.)

Wilson's General Report appears on pp. 137 of the State-of-the-Art Volume.

Président A. MAYER

Je remercie vivement notre Rapporteur Général pour son très remarquable rapport et non seulement pour la clareté de son exposé, mais aussi pour le soin qu'il a pris de rester dans les limites de temps prévues.

Je voudrais maintenant passer la parole aux Membres du Bureau sur certains points particuliers que nous avons choisis d'accord avec le Rapporteur Général comme présentant un intérêt pour vous tous.

Et tout d'abord le Professeur Casagrande, Ancien Président de cette Société, qui va nous parler du traîtement des fondations de grandes profondeurs. Je n'ai évidemment pas besoin de vous présenter le Professeur Casagrande et je lui donne tout de suite la parole.

Panelist A. CASAGRANDE (U. S. A.)

SYNOPSIS

This paper is intended as a discussion of the chapter on "Seepage Control" in the report on EARTH AND ROCKFILL DAMS by Stanley Wilson and R. Squier. Additional examples of previous foundation treatment are described, some in the design stage, others under construction or already completed, including some historical comments on the early development of various treatments.

INTRODUCTION

In their monumental state-of-the-art report on EARTH AND ROCKFILL DAMS, Stanley Wilson and R. Squier have included an excellent and well documented review of methods for controlling seepage through pervious alluvium. Therefore, my task is greatly facilitated and will consist of some supplementary comments with the aid of additional examples of pervious foundations for dams, some completed, others under construction or still in the design stage.

EARLY EXAMPLES OF CONCRETE CUTOFF WALLS CONSTRUCTED IN THE THIRTIES

The Quabbin Reservoir, in the western part of Massachusetts, which provides the main source of water to the city of Boston, was created by two hydraulic fill dams which were built in the Thirties. Investigations, design and construction of the first of these two dams are well described by Stanley M. Dore (1935). Cutoffs had to be constructed through sand and gravel, containing a large number of cobbles and boulders, with a maximum thickness of 150 ft. After sinking an exploratory pneumatic caisson to bedrock, the designers decided to build a cutoff wall by first excavating a cut with sloping sides to an economical depth, assisted by lowering the groundwater table, and then by sinking reinforced concrete caissons end to end and sealing them with concrete to each other and to bedrock, thus forming a positive cutoff along the center line of the dam. (The same design and construction procedure was also adopted for the second dam.) Each caisson was 45 ft long, 9 ft wide, and had three 4 ft diameter access wells. Between the ends of adjoining caissons an 18 in. space was left which together with the keyways in the end walls of the caissons created a working space with a nominal width of 4.5 ft for excavating the closure section.

The caissons extended 25 ft above the bottom of the open-cut trench to provide proper embedment in the core. With this surcharge, the dead load of each caisson was sufficient to sink it, supplemented only occasionally by "blowing" or the use of dynamite. After the cutting edge reached bedrock, the working chamber was "underpinned" by extending 1.5 ft thick walls all around to bedrock. Construction of this "underpinning chamber" was carried out under compressed air. After cleaning the bedrock surface, concrete was poured to an elevation above the cutting edge; then the air pressure was released, grout curtain holes were drilled and the grouting operations were carried out from the working chamber, which was finally filled with concrete.

The openings between caissons were excavated to the groundwater surface by using horizontal wooden sheeting. Then a concrete plug, containing a 3 ft diameter access well, was poured. Excavation in each keyway was then continued below groundwater level by underpinning methods, under compressed air, with 1.5 ft walls carried down along the faces of adjoining caissons. Finally the entire space was filled with concrete.

Pumping was carried out continuously during the sinking of caissons and the keyway closure construction, with the groundwater surface lowered as much as 90 ft below its original level. Thereby it was possible to carry out 70% of the work without use of compressed air and 20% under pressures less than 10 psi.

The cutoff wall for the first of the two dams (which was completed when Stanley Dore wrote his paper) cost $940,000 for a total wall area of about 140,000 sq ft. Today, such a wall would cost at least five times as much; but even such an amount would be comparable to the cost per square foot of a concrete wall consisting of individual pile elements of the ICOS type as executed, for example, for the upstream cofferdam at the Manicouagan 5 Dam. Therefore, it would be of interest to carry out a comparative cost estimate using today's prices (1) for the Quabbin Dam walls as built, and (2) for a 2 ft thick pile-type wall. Of course it need not be stressed that caisson-type concrete walls, such as executed for the Quabbin dams, constitute a much more positive cutoff than a single 2 ft thick concrete wall, particularly when constructed by the panel method.

Use of such pneumatic caissons would usually be limited to conditions where the groundwater level could be lowered progressively to an extent which would eliminate the need of air pressure for most of the work. Also, the presence of many large boulders would probably render caisson walls excessively expensive. The costly and time-consuming closure sections could probably be replaced by ICOS-type piles. Furthermore, such piles could also be used adjacent to steep rock walls; and any overhangs could be treated by grouting. With such modifications it would seem that caisson-type cutoff walls would deserve serious consideration also under present conditions.

In connection with this discussion I should mention that after completion of the two Quabbin dams the Board of Water Supply of the City of New York built two earth dams with similar cutoffs, i.e. combining a deep open excavation and a concrete wall consisting of interconnecting pneumatic caissons which extend to bedrock.

In the Thirties, Karl Terzaghi served as consultant on the Bou-Hanifia Dam (in some references spelled Bu-Hanifia) in Algeria, a high rockfill dam with an impervious upstream facing which along the upstream toe is continued in the form of a 4 meter wide concrete cutoff wall through pervious, very soft sandstone strata to a maximum depth of 70 meters, into an impervious stratum of marl. Because of anticipated settlements from consolidation of the marl, vertical joints with a spacing of 15 to 20 meters, and with special water stops, were installed in this cutoff wall. In addition, Terzaghi provided along the base of the rockfill dam a system of filter layers, to ensure protection against piping, in case the cutoff wall should develop serious defects. It is worth recalling that the well-known Terzaghi criteria for the design of filters were developed in connection with this project.

The foundation conditions, design and construction of the Bou-Hanifia dam and of several other similar rockfill dams in Algeria were described by Drouhin (1936), Gutmann (1937, 1938) and Falconnier and Lombard (1942). (As an interesting sidelight: In his 1937 paper, Gutmann mentioned that the failure of a "loose rockfill dam" had convinced French engineers of the importance of placing rockfill well compacted. Quoting Gutmann: "What is dead insofar as French engineers are concerned, is the practice of loose-dumped rockfill; the fill of all the new dams has been carefully placed by machine and by hand. Furthermore, all these dams are provided with heavy cutoff walls carrying inspection tunnels for their entire length... ".)

DESIGN STUDIES FOR VERY DEEP CONCRETE CUTOFF WALLS

For the damsite Manicouagan 3, which is located downstream of the Manicouagan 5 Dam in Canada, design studies are being carried out by Hydro-Quebec and their engineers, Asselin-Benoit-Boucher-Ducharme-Lapointe, for a 350 ft high earth dam which is underlain by a 400 ft deep rock gorge that is filled with pervious alluvium heavily infested with boulders in the lower portion. Use of an impervious upstream blanket would be uneconomical because it would require greatly increasing the length of very large diameter diversion tunnels. Furthermore, there is a shortage of suitable impervious material. For these reasons detailed studies were carried out of multiple row grout cutoffs and of concrete wall cutoffs. One of the proposals being studied is illustrated in Fig. 1. It comprises two parallel concrete walls which in the deep portion would consist of ICOS-type pile elements, i.e. the same type of construction as was used for the 250 ft deep cutoff wall at Manicouagan 5 Dam, and which is described by Galbiati (1963). This type of wall is made of 2 ft wide alternating round and interlocking "piles" The 2 ft round piles are installed first, and the holes for 2 ft wide interlocking piles are drilled with a special chisel with expanding, scalloped-shaped wings which follow the round piles. Such a pile-type wall can be installed in ground containing large boulders, in contrast to a panel-type wall. Also, in the installation of such 2 ft wide pile elements the danger of a cave-in is practically eliminated, whereas there is always a risk that the sides in a panel-type wall may cave locally while concrete is being placed, with the risk of creating, unseen, a serious discontinuity in the wall.

The fact that a pile-type wall was successfully installed at Manicouagan 5 to a depth of 250 ft was not sufficient evidence that such a wall could also be installed in similar alluvium to a depth of about 400 ft. Therefore, Hydro-Quebec engaged Icanda and Soletanche to carry out independent tests at that site. Each contractor, using different equipment, successfully installed two round piles and one interlocking pile to and into bedrock at a depth of about 400 ft, thus proving that it would be feasible to construct this type of cutoff wall to such an unprecedented depth.

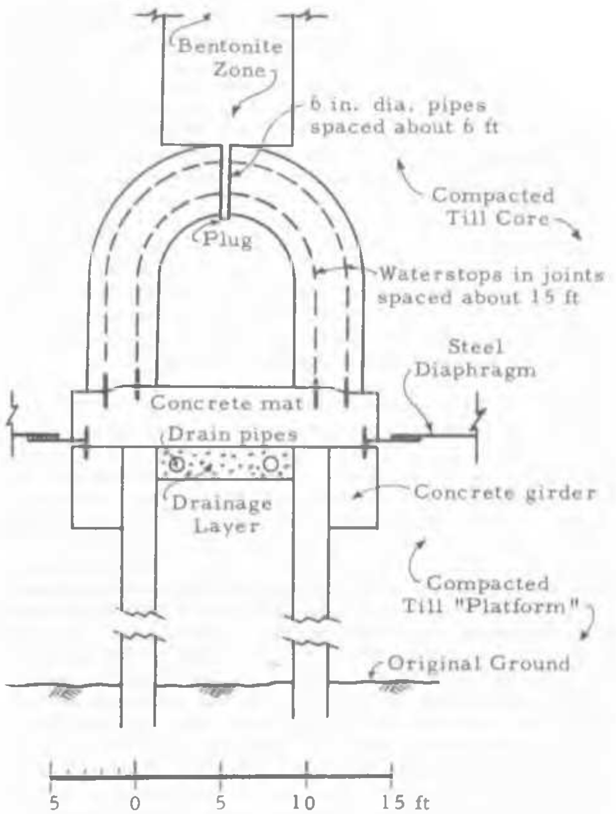


Fig. 1 Proposal for Double Wall Cutoff with Observation Gallery

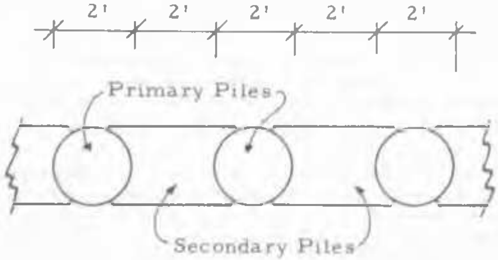


Fig. 2 Pile-Type Cutoff Wall

The reasons for proposing a double wall, topped by an observation gallery, and for other details shown in Fig. 1, are summarized as follows:

1. As proposed, the cutoff walls would be located along the centerline of the dam, beneath a wide core, with the upper 30 ft extending through the "working platform" which would consist of a 30 ft thick zone of core material. Thus, the path of percolation around the top of the cutoff would be ample. Nevertheless, the high stress concentration which the walls and gallery will produce in the bottom of the core will tend to cause formation of cracks in the core zone surrounding the gallery. By means of numerous piezometers in that zone one would be able to detect such conditions and correct them by grouting. As an additional line of defense against water flowing through a system of cracks through the till platform and over the gallery, horizontal impervious membranes would extend about 50 ft upstream, and to a lesser distance downstream, at the elevation of the top of the walls. It is suggested to make these membranes of overlapping 1/4 in. thick stainless steel sheets which are bedded in neoprene gaskets. Transverse butt joints would be covered with wide strips of sheet steel and with neoprene gaskets. Such a membrane would be able to bridge effectively across open cracks and would have ample flexibility and stretch to adjust itself to horizontal strains without tearing.

2. To eliminate high stress concentration on top of the gallery, and to control the stresses within desirable limits, a zone of bentonite, with a consistency approximately at the plastic limit, is placed above the gallery, about 8 ft wide and 20 ft high. The compressibility of this zone by itself would prevent high stress concentration on top of the gallery. As additional means of control, 6 in. diameter pipes penetrate the roof of the gallery. By opening the bottom plug, one could "bleed" bentonite from that zone. Probably the bentonite would not actually flow, and one would have to remove a small quantity with an auger. By means of pressure cells on the outside of the gallery one would observe the vertical and horizontal stresses on the gallery and monitor the effect of bleeding bentonite from above the gallery. (Note: Hydro-Quebec has succeeded in developing an effective method for mixing and placing granular bentonite at the plastic limit. Tests at various water contents have shown that the greatest dry unit weight of the compacted bentonite can be achieved with water content very close to the plastic limit.)

3. Instrumentation within the walls, particularly strain gages, would be used to determine the crack pattern. I believe that just as soon as a crack develops, the compressive stresses in the adjoining portions of the wall will relax, so that the actual displacement along a shear crack would be minor. Then the build-up of compressive strains would produce a crack at another elevation. Thus, one may expect a series of shear cracks, but with very small displacement along each crack. Such a system of cracks would remain for practical purposes watertight.

4. The cutoff walls would be arched in an upstream direction, following the arched alignment of the dam. When the water load is applied against the upstream side, arching action would tend to increase substantially the compressive stresses within the vertical joints between the pile elements of the wall; and these forces would tighten the joints against leakage through the joints.

5. From observations of the performance of the walls during construction of the dam, and from piezometer observations, it would be possible to determine if and where grouting should be carried out between the concrete walls.

Similar to the wall installation at Manicouagan 5, one would drill the pile units at least 2 ft into bedrock. In addition, casings would be installed in every second pile through which core borings would be made into the rock for detailed exploration and grouting. Should it develop that not all pile units could be advanced to the full depth of this rock canyon, then the casings would be used for the purpose of drilling grout holes and grouting the remainder of the alluvium. Since the lowest part of the rock canyon is very narrow, this would involve only a very small portion of the entire cutoff area.

DEEP OPEN-PIT EXCAVATIONS FOR CUTOFFS

Wilson and Squier have emphasized in their state-of-the-art report, that "a positive cutoff, formed in an open excavation to an impervious stratum and which is backfilled with compacted impervious material is the most desirable form of cutoff". In my practice I have noticed that many designers have an exaggerated fear of the difficulties and costs of deep excavations. It is chiefly the uncertainties of controlling the seepage into the excavation which discourage the designer. To assist them in their task, they would need a comprehensive review of deep excavations for cutoffs that have been successfully executed, including a detailed description of the overburden in which the excavation was made, and with an analysis of difficulties encountered. Such a study should include not only deep excavations for earth and rockfill dams but also for concrete dams. For example, the 235 ft deep excavation for Parker Dam (Eng'g News-Record, 1937) on the Colorado River, which in 1937 was probably an unprecedented depth of excavation for any dam, is a very instructive example of a deep excavation through highly pervious overburden.

An exceptionally deep excavation, about 260 ft from riverbed, for a trench cutoff is now in progress for the Lower Notch Dam in Canada, Fig. 3, which was designed by H. G. Acres & Company for the Hydro-Electric Power Commission of Ontario, assisted by D. H. MacDonald, F. A. Nickell, R. B. Peck and the author as Board of Consultants. This rock valley is filled in the upper part with lacustrine, stratified silts and find sands, and the lower part, a narrow rock gorge, contains a highly pervious coarse-grained stratum which is under artesian pressure.

A number of different solutions for the control of seepage through the foundation were investigated, including the use of an impervious blanket, a grout curtain, and various concrete wall cutoffs, all starting from the riverbed, and also combinations of open trenches excavated to various depths and continued by grouting or some type of wall cutoff. Since the side slopes would have to be much flatter, and the diversion tunnel longer, for a dam resting directly on the lacustrine stratum, it developed that the cost estimates for all schemes were approximately the same. After verifying, by means of extensive pumping tests, that it would be practicable to execute an open trench excavation to the full depth, the decision in its favor was easily made. As can be seen in Fig. 3, groundwater control during excavation will be effected by tube wells on the inside of the upstream and down-

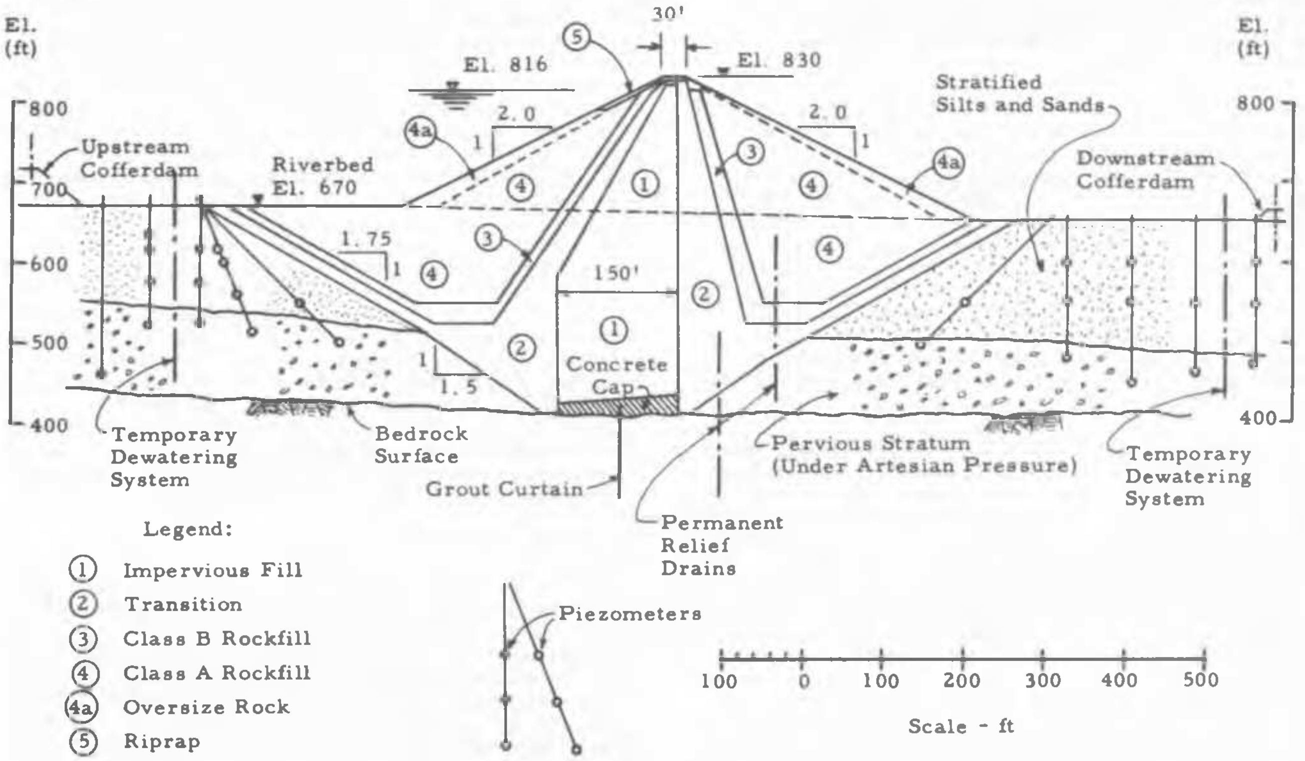


Fig. 3. Lower Notch Dam

stream cofferdams, and by means of numerous piezometers.

FOUNDATION TREATMENTS WHICH ALLOW SEEPAGE

The single sheetpile wall beneath the Fort Peck Dam was intended to form a reasonably effective cutoff. But in 1943, when the drop in head across that wall was measured, it was found to be only 10 to 15% of the total head difference; Lane and Wohlt (1961). In the preceding year, 1942, the piezometric level in the lower sand stratum along the downstream toe had risen to 45 ft above ground surface; and then relief wells were installed which effectively reduced that dangerous excess head to a tolerable amount. This experience led to a systematic use of relief wells along the downstream toe of dams whenever there was any doubt in the designer's mind that the measures for controlling underseepage could be relied upon. It also supported the doubts that many practicioners had about the effectiveness of sheetpile walls as cutoffs. However, others continued to believe that they were effective and insisted on their use whenever a complete open-trench cutoff was not feasible. Such opinions also divided the Board of Consultants for the large dams on the Missouri River which the Corps of Engineers was designing downstream of the Fort Peck Dam, namely the Garrison, Oahe, Fort Randall, Gavins Point and Big Bend Dams. Together with the late L. F. Harza, I belonged to a minority that proposed a wide impervious upstream blanket in lieu of the sheetpile wall, supplemented by relief wells. A majority favored a compromise solution, i.e. a shorter impervious blanket combined with the sheetpile wall. (In retrospect, I am now in sympathy with the views of the majority because there was then not sufficient empirical knowledge available for a reliable design and execution of measures for controlling underseepage.) And so the Garrison and Oahe dams were built with a combination of an impervious blanket, a sheetpile wall and relief wells near the downstream toe. During the early years of operation the measured head loss across the sheetpiling ranged between 8% and 18% of the hydraulic head at Garrison Dam, and between 6% and 23% at Oahe Dam. The cutoff efficiency of these sheetpile walls gradually increased in the course of years, as described by Lane and Wohlt (1961).

When the final decision concerning measures for the control of underseepage had to be made for the Fort Randall Dam, an influential member of the Board, who strongly advocated sheetpile walls, happened to be absent. The much greater thickness of pervious alluvium at this site, with a maximum depth of 170 ft, may have been an important reason why the other members hesitated. Perhaps I was more outspoken at that meeting. Whatever the reasons, the same consulting board agreed to omit the sheetpile wall at Fort Randall which had been included already on the design drawings. As criterion for the length of the impervious blanket I proposed that the distance from the upstream edge of the blanket to the downstream toe of the dam should be about 20 times the maximum hydraulic head. In addition, relief wells were installed which drained into a highly pervious waste berm of chalk.

When the last two of the dams on the Missouri were designed, the Gavins Point and Big Bend Dams, it was also decided to control underseepage by a combination of an impervious upstream blanket and downstream relief wells.

The satisfactory performance of the Fort Randall, Gavins Point and Big Bend Dams established a very conservative precedent for the length of impervious blankets which later guided the design of other dams. But the high cost of the blanket as compared to the very low cost of relief wells, created a trend toward installation of more comprehensive systems of drainage wells combined with a reduction in the length of the impervious blanket. An outstanding example is the 465 ft high Tarbela Dam on the Indus River, in West Pakistan, which is now under construction. This dam was designed by Tippetts-Abbett-McCarthy-Stratton for the West Pakistan Water and Power Development Authority. Fig. 5.

The pertinent dimensions of the seepage control measures at Tarbela Dam are as follows. The distance from the upstream edge of the impervious blanket to the downstream toe is almost 7000 ft, which is approximately 15 times the hydraulic head. The impervious blanket decreases from a thickness of 40 ft at the upstream toe of the dam to 5 ft at the upstream edge of the blanket. The entire downstream shell of the dam is underlain by a drainage layer. The wells near the downstream toe are spaced 50 ft on centers. They extend to a depth of 125 ft, except every 8th well which is 250 ft deep. The wells discharge into a drainage gallery from where the water is led by means of conduits through a berm. The alluvium, consisting of sands, gravels and cobbles, which extends to a maximum depth of over 600 ft, would have rendered the expense for any kind of positive cutoff prohibitive.

In contrast to the Tarbela Dam, where I agreed to a reduction in the length of the impervious blanket, I found it necessary to recommend for the Arrow Dam, on the Columbia River, a greater length than the precedent established on the Missouri Dams. This dam, which after its completion was named Keenleyside Dam, was designed by C.B.A. Engineering for the British Columbia Hydro and Power Commission. Similar to the wide Indus River at Tarbela, the wide Columbia River at the Arrow Dam site is underlain by pervious alluvium to a maximum depth of over 600 ft, and it would not have been feasible to construct a positive cutoff of any type. Furthermore, the great depth of this river, even at low water stages, made it necessary to build the lower portion of the dam and the impervious blanket by dumping through water. In particular one could not be certain that a fully satisfactory impervious blanket could be achieved in this manner. Therefore, the consultants recommended an exceptionally conservative creep ratio for the length of this blanket. As actually constructed, Fig. 4, the distance from the upstream edge of the blanket to the downstream toe of the dam is 2850 ft, as compared with a maximum hydraulic head of 76 ft. In addition, the design included installation of relief wells, but they were to be installed only if the piezometer observations indicated that the impervious blanket was defective. However, observations are demonstrating clearly that the blanket is functioning perfectly, and that there is no need for the relief wells. For a general description of the Arrow Project see Henry and Grant (1967), and concerning the soil mechanics features of this dam see Golder and Bazett (1967).

CLOSING REMARKS

There are very few aspects of earth dam engineering which can be evaluated with the same degree of confidence as the design of steel or concrete structures. The measures for controlling seepage through pervious foundations range between the extremes of reliable and highly uncertain measures. Depending on the degree of uncertainty or calculated risk involved, one is forced to use a second, and sometimes a third line of defense. As Terzaghi has often emphasized, the designer must make the worst assumptions that are compatible with the geologic conditions. To this should be added, that he should also assume the worst conditions that can develop during construction. After many years, the profession has reluctantly learned that sheetpile walls and single-line grout curtains form very poor cutoffs. But how certain can we be of the various types of concrete walls, or slurry trench cutoffs? How much of a second line of defense should one provide? Relief wells form a reliable and inexpensive second line of defense and should be used whenever there remain any doubts about the reliability of the principal measure for con-

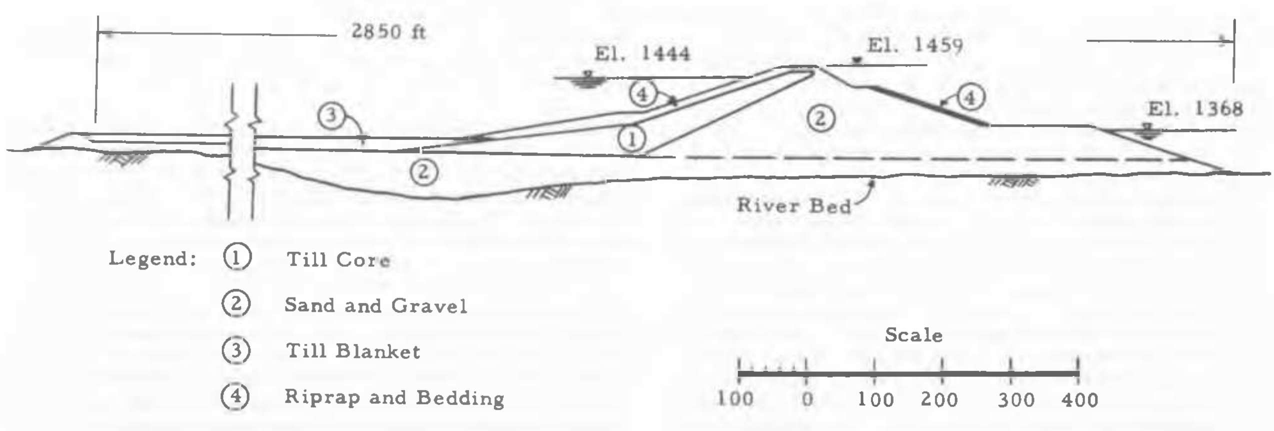


Fig. 4 Arrow Dam - Recently named Keenleyside Dam

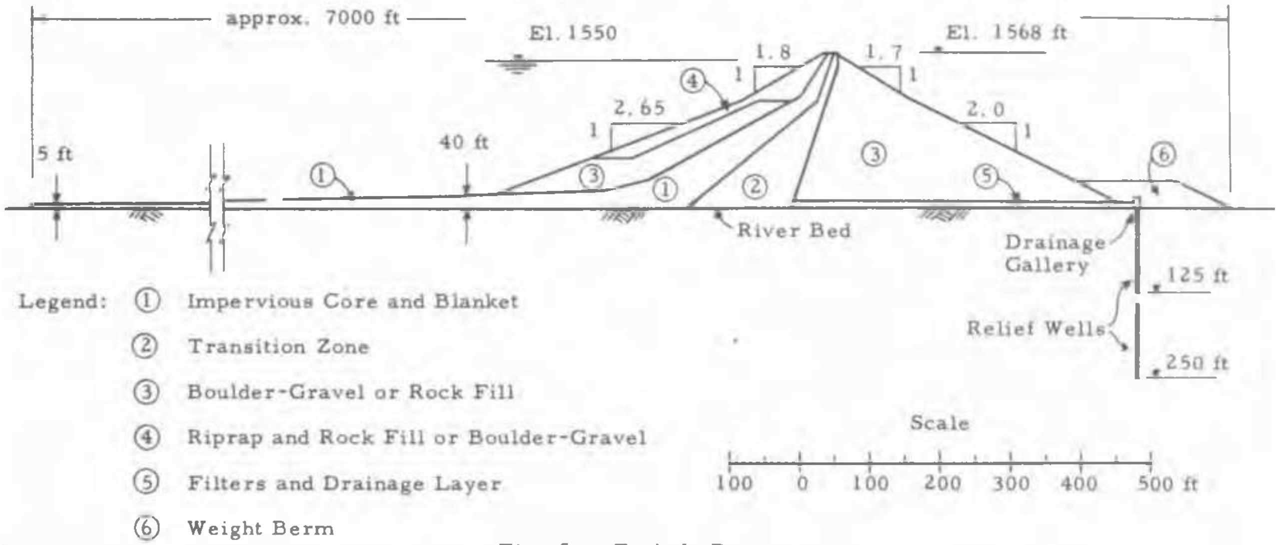


Fig. 5 Tarbela Dam

trol of seepage.

Fortunately it is simple and inexpensive to monitor the performance of seepage control measures. Most dams for which foundation seepage is a problem are appropriately instrumented and valuable information on the performance of the measures to control seepage is accumulating in files. We need now a concerted effort to systematically analyse these data and to develop useful guide lines for the designers.

REFERENCES

Dore, S. M., "Design and Progress on Construction of Dams for Quabbin Reservoir," Journal, Boston Society of Civil Engineers, Vol. 22, July 1935, pp. 151-177.

Drouhin, M., "Essais geotechniques des terrains de fondation," Transactions Second Congress on Large Dams, Washington, D.C. 1936, Vol. IV, pp. 3-66.

Engineering News-Record, "Deep Excavation for Parker Dam," Nov. 25, 1937, pp. 853-856.

Falconnier, A., and Lombard, A., "Etude geologique des terrains de fondation du Barrage de Bou-Hanifia," Bulletin Technique de la Suisse Romande, June 13, 1942.

Galbiati, I.V., "Concrete Cutoff Wall Drilled Down 250 ft," Civil Engineering, Nov. 1963, pp. 59-61.

Golder, H.Q., and Bazett, D.J., "An Earth Dam Built by Dumping through Water," Transactions Ninth International Congress on Large Dams, 1967, Vol. IV, pp. 369-387.

Gutmann, I., "Algerian Dams of Placed Rockfill," Engineering News-Record, Dec. 2, 1937, pp. 889-894.

Gutmann, I., "Algerian Rockfill Dam Substructures," Engineering News-Record, May 26, 1938, pp. 749-751.

Henry, K.A., and Grant, P.N., "Conception, Investigation and Design of Arrow Dam," Journal of the Power Division ASCE, Vol. 94, No. P01, May 1968, pp. 41-65.

Lane, K.S., and Wohlt, P.E., "Performance of Sheet Piling and Blankets for Sealing Missouri River Reservoirs," Transactions Seventh International Congress on Large Dams, Vol. 4, 1961, pp. 255-279.

Président A. MAYER

Je remercie vivement le Professeur Casagrande pour ses très intéressantes observations et je voudrais passer la parole au Professeur Marsal sur un problème particulier de fondations. Je n'aurai pas non plus besoin de présenter le Professeur Marsal ici à Mexico, où tout le monde le connaît et connaît ses travaux remarquables.

Panelist R. J. MARSAL (México)

The design of an embankment on a fissured clay shale foundation has aroused doubts about the shear strength that these materials may develop. For the present example, conventional triaxial tests were initially run on undisturbed specimens. The relatively high shear strength values obtained upon testing a few samples that survived the indeed troublesome field and laboratory handling, induced to perform large scale, direct shear tests at the site. A discussion of test results and provisions taken for the construction of the dike, as well as a brief account of its performance during five years, is presented.

Site Geology. Dike No. 2 is located in one

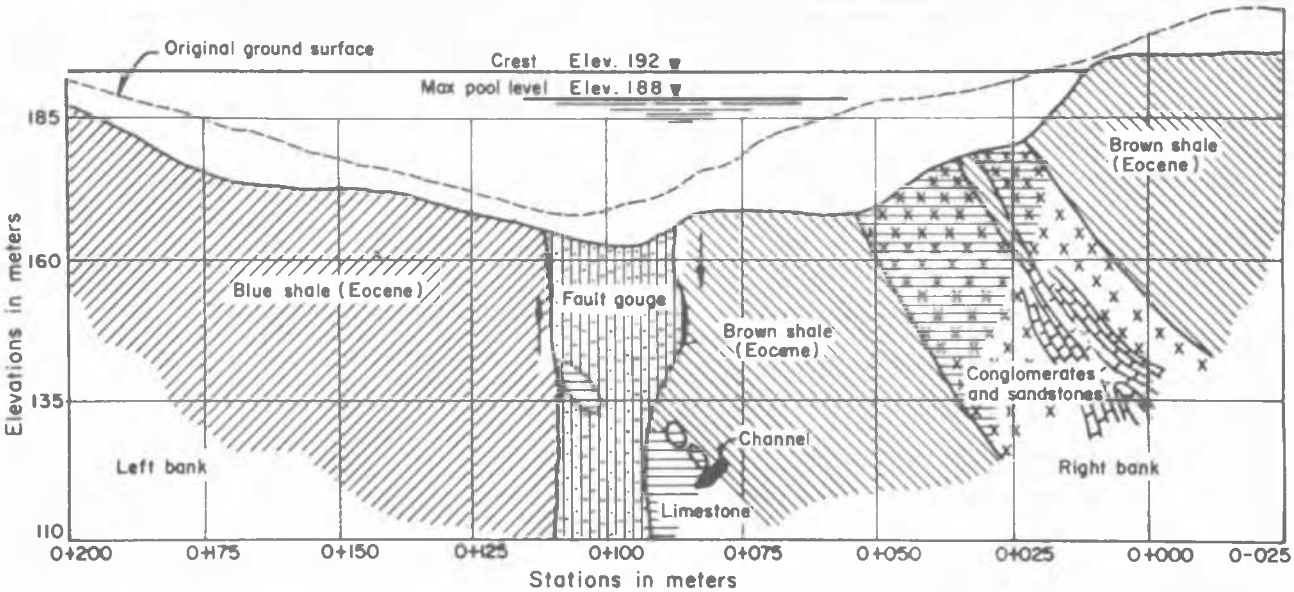


FIG 1 GEOLOGICAL PROFILE ACROSS THE DIKE

of the saddles surrounding the reservoir of Netzahualcóyotl dam built by the Secretaría de Recursos Hidráulicos in South-East Mexico. Geological explorations revealed that this saddle is crossed by an important fault, which put into contact different shales of the Eocene (Fig. 1). The axis of the dike is approximately normal to the fault; the gouge zone is about 10 m wide. Towards the right abutment, silty and fissured shales of brown shade are found; the left abutment is made of intensely fissured blue shales. Fig. 2 shows the position of remolded samples of these rocks in the plasticity chart.

Of the two shales, the blue one on the left abutment arose the most serious doubts as a foundation material for the dike, since it is brittle and breaks easily through slickensides. After remolding, it has liquid and plastic limits of 35 and 20 percent, respectively. The specific gravity is 2.75 and the dry unit weight about 2.1 ton/m^3; thus, the void ratio in natural state results 0.31.

The most outstanding characteristics of the brown shale on the right abutment is its high permeability, judging from grout takes at the axis of the dike. Besides, in this bank at elevation 150, remnants of karstic limestone were encountered both in exploratory bore holes and in the drainage gallery, excavated downstream from the axis of the dike. Seepage of water in the blue shale is better distributed and discharges are smaller.

The fault is filled with very heterogeneous materials; there are sand pockets surrounded by highly plastic clay of various colors.

<u>Shear Strength of Fissured Shales</u>. In order to determine the shear strength of the fissured materials found at the site, a torsion-shear apparatus of large dimensions was designed and built by the Comisión Federal de Electricidad (Fig. 3), in cooperation with the Secretaría de Recursos Hidráulicos.

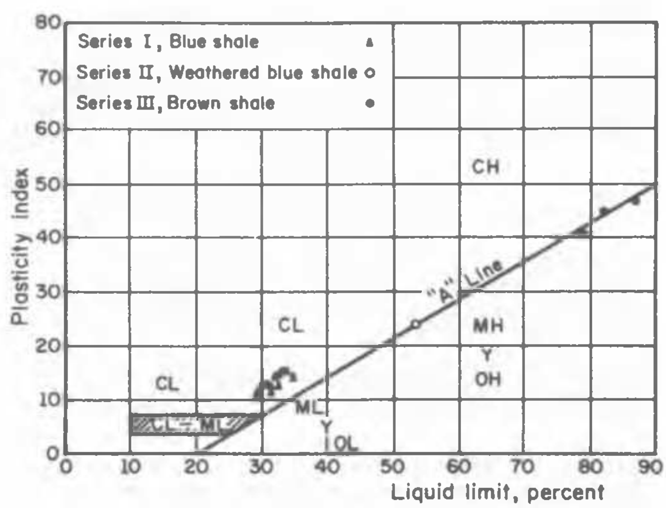


FIG 2 PLASTICITY CHART

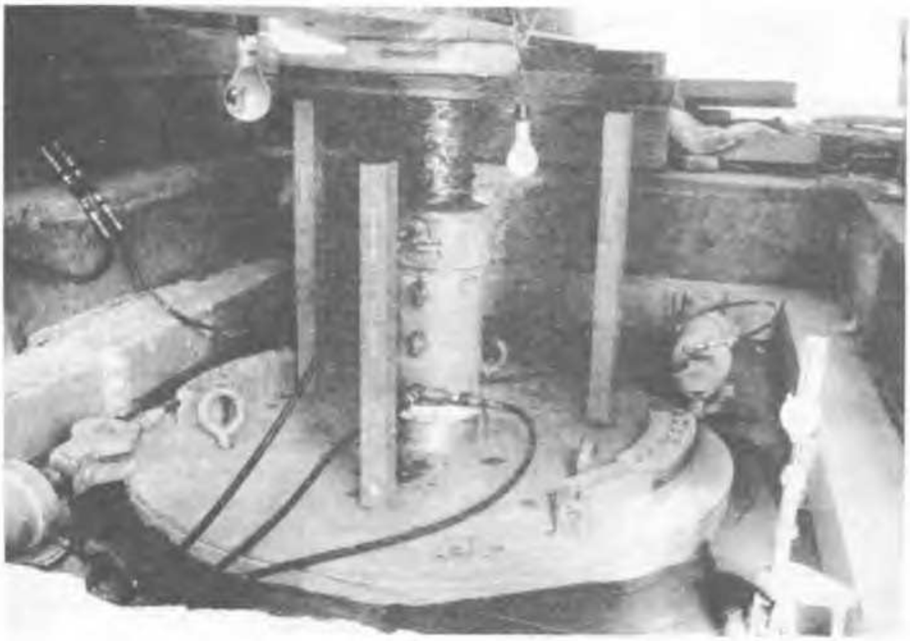


FIG.3
TORSION SHEAR APPARATUS. AT THE CENTER, DISC, HYDRAULIC CYLINDER FOR APPLYING NORMAL LOADS, AND GRADUATED SCALES; AT LEFT FOREGROUND AND RIGHT BACKGROUND, HORIZONTAL JACKS TO APPLY TORQUE. NOTICE THE LOADING FRAME, AND DIAL EXTENSOMETER CONNECTED TO THE DISC BY A CABLE.

The above apparatus consists of a steel disc 160 cm in diameter and 6.5 cm thick, on which blades have been welded, forming 72 boxes 7x10x1.5 cm at the periphery; the ring-shaped shearing area is about 0.5 m^2. The torsion couple is transmitted to the disc by means of two steel cables acted by horizontal hydraulic cylinders that operate in tension, each one with a capacity of 10 tons. These jacks react on a metallic frame, which is laid horizontally inside the excavation made previously at the testing site. Normal loads are applied to the disc by means of a vertical hydraulic cylinder, 200 tons in capacity, acting on a thrust bearing to minimize friction during rotation; the jack in turn thrusts on a ballasted platform. Both normal loads and torsion couples are computed from pressures measured at the respective cylinders with calibrated Bourdon gages. The rotation of the disc is recorded by means of an Ames extensometer with an accuracy of 0.01 mm and a total range of 8 cm.

Series of tests were performed at two locations on the left abutment (blue shale) and at one of the right bank (brown shale). Normal pressures varied from 0.5 to 6 kg/cm^2 and increments of the torsion couple of about 10% of the failure value were applied at 5 minutes, 1 hour and 6 hours intervals in each series, respectively. The material was saturated under the own weight of the apparatus and further consolidated under the predetermined normal pressure, before testing it in shear. Deformations developed were large enough to determine the residual shear strength of the shales. Shear stress vs tangential displacement curves for the blue shale tested under a normal load of 3.88 kg/cm^2 and for different shearing rates, are shown in Fig. 4. The influence of the rate of shearing was not significant in any of the materials tested. Fig. 5 exhibits the residual shear strength in terms of the normal pressure for the weathered and the fresh blue shales as well as that of the brown shale. Results fall within two straight lines with the following equations:

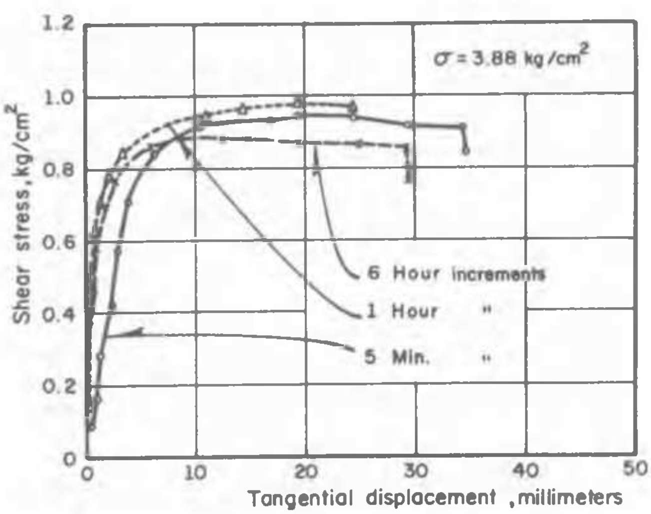


FIG 4 TORSION SHEAR TEST ON THE WEATHERED BLUE SHALE (EFFECT OF THE SHEARING RATE)

1) upper envelope, $s=0.3+0.16\sigma$ (kg/cm^2);
2) lower envelope, $s=0.16\sigma$.

Design of the Dike and Instrumentation. Based on the above characteristics of the foundation, it was decided to verify the stability of the dike assuming a shear strength, $s=0.16\sigma$, under static loads, and $s=0.2+0.16\sigma$ (kg/cm^2), for stress conditions including seismic forces. The wedge method of analysis was used and a homogeneous type of embankment with a central vertical drain recommended. This drain, 1 m thick, made of well graded sand compacted in layers, is connected to a drainage blanket, 1.5 m in thickness, which covers the downstream base of the dike. Fig. 6 presents the principal features of this structure as built. Note that underneath the dike, 24 m downstream of its axis and at Elev. 154, a drainage gallery extending well into both abutments was excavated. This gallery is concrete - lined andhas drain wells cased with 2 inches perforated plastic pipes, filled with sand. The upward drainage wells reach to within 3 m of the ground surface, and the downward ones extend 25 m below the bottom of the gallery. To relieve neutral pressures in the downstream portion of the foundation, 6 inch wells, 10 m deep, were drilled; they were filled with coarse sand and connected to the above mentioned drainage blanket.

The body of the dike, except for the outer rockfill protections and the vertical drain, was built with the weathered brown shale borrowed from the hills close to the site, on the right abutment. As shown in Fig. 2, the soil is a clay having a plasticity index that varies between 40 and 50. It was placed with the natural water content, considerably above the optimun.

The instrumentation that includes surface monuments, inclinometers, linear extensometers and piezometers is shown in Fig. 6, both in plan and cross section. Note that most of the instrumentation is intended to monitor the performance of the foundation.

Performance. The embankment was completed in November 1964, and frequent measurements have been made in all instruments from the start of construction onwards. Results obtained during 1968 are presented in Fig. 7.

Piezometers in the upstream portion of the foundation have shown a steady drop, from their installation in 1964 to the end of 1965. At this time they reached a stable piezometric level, this being apparently independent of the water elevation in the reservoir (Fig. 7, d). The maximum pool level stood at Elev. 177 in January 1967. Downstream, foundation piezometers show a static head corresponding approximately to the elevation of the drainage blanket or lower. Pore pressures within the embankment are relatively low.

Inclinometers were installed in bore holes during the construction of the embankment. Some of these showed at the begining small erratic movements, probably the consequence of adjustments of the casing. After adopting in August 1965 a new origin for each casing, movements in the foundation were detected, particularly in inclinometers III and V (Fig. 7, a). These horizontal displacements indicate that the construction of the embankment may be causing movements on both the left abutment and the gouge zone. Surveys of surface movements reveal a displacement pattern which does not conform with that shown by the inclinometers; this may confirm that the above movements developed in the foundation, below Elev. 160.

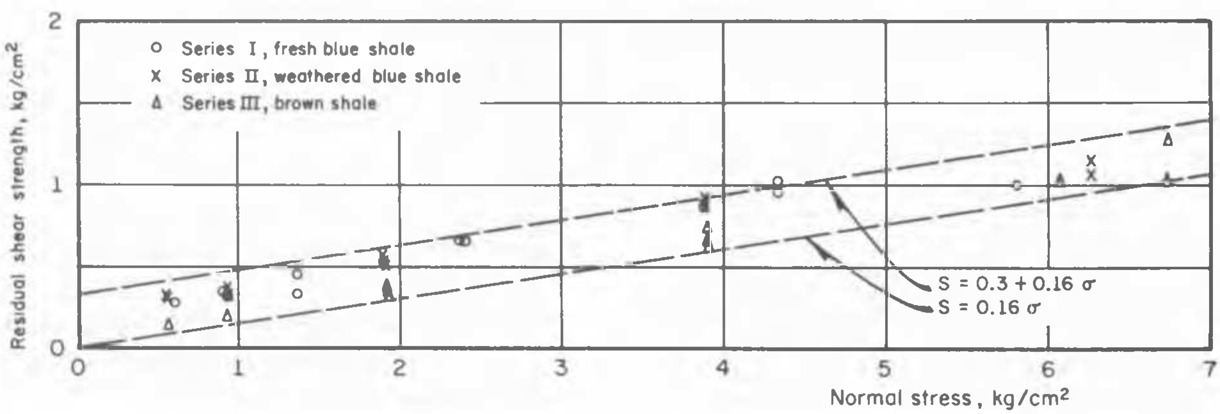


FIG 5 RESIDUAL SHEAR STRENGTH OF THE BLUE AND BROWN SHALES

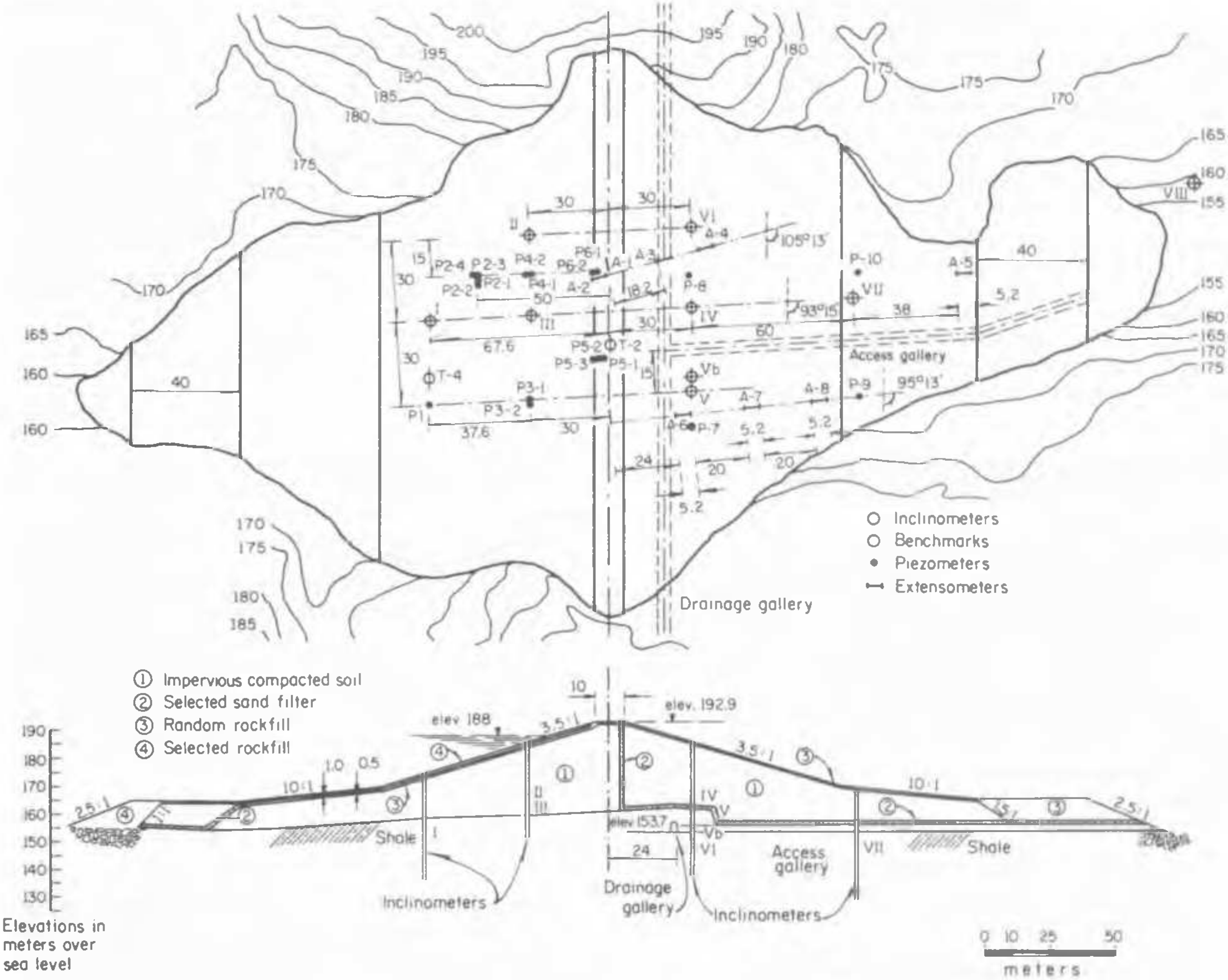


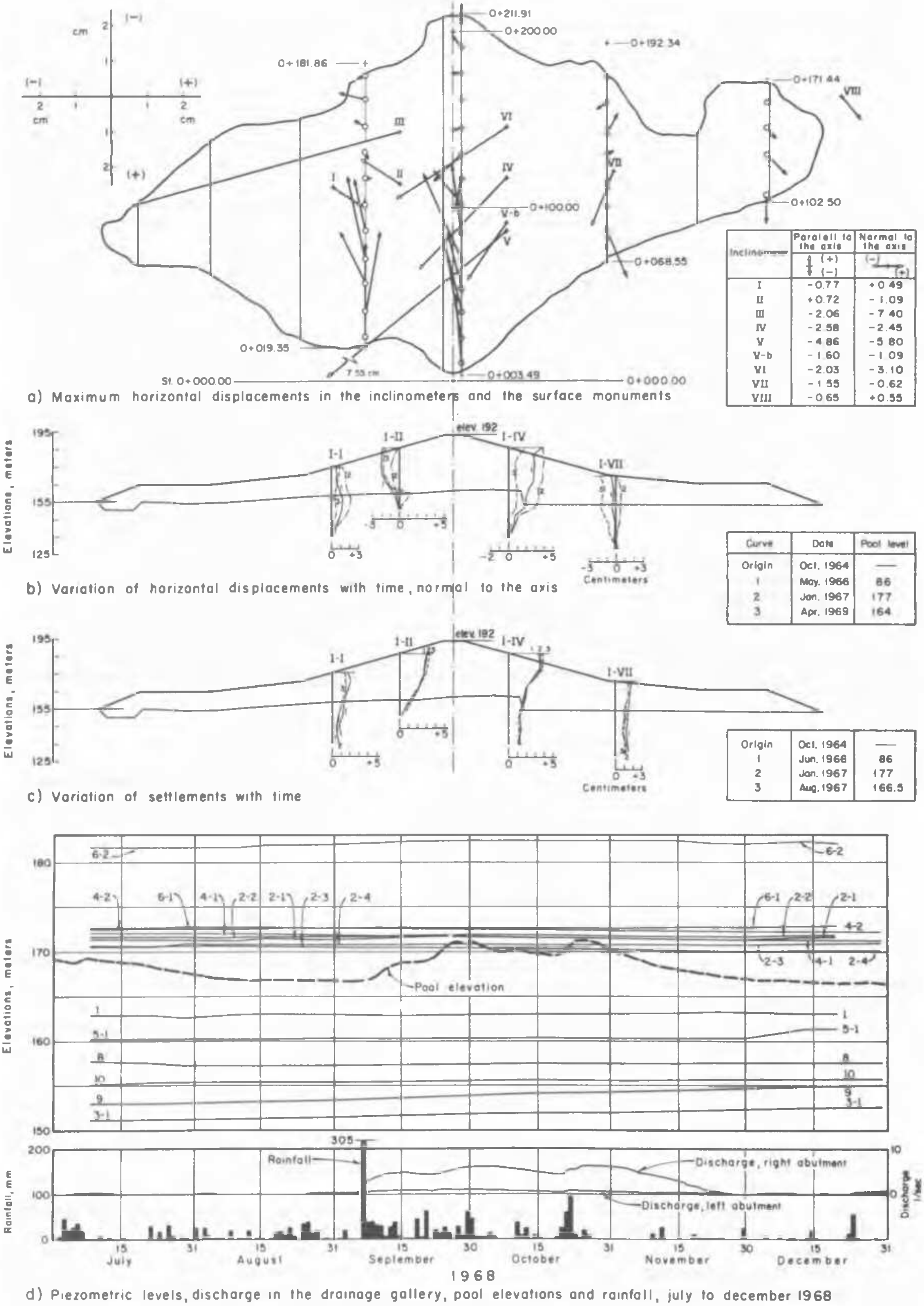
FIG 6 DIKE No 2 MAXIMUM CROSS SECTION

Variations of horizontal displacements and settlements along inclinometers I, II, IV and VII, in three different dates, are plotted in Fig. 7, b and c. Horizontal displacements up to January 1967 increased towards downstream; this tendency has reversed since then. Settlements measured in the above inclinometers are moderate, both in the foundation and the embankment, the total maximum values being smaller than 5 cm.

Horizontal extensometers were installed at several locations in the downstream portion of the foundation. During construction, strains of about 0.15 percent have been recorded in four of the measuring devices, the other four showing extensions of less than 0.05 percent.
Horizontal movements and settlements measured at surface monuments have been very small after completion of the embankment. Horizontal displacements are less than 3 cm and the maximum settlement is about 4 cm (Fig. 7, a).

Seepage through the drainage system of foundation gallery appears to be related to rainfall (Fig. 7, d). In the dry season the leakage drops to zero, whereas it reaches maximum values of 8 l/sec during the rainy season, the right branch of the gallery producing about 80 percent of the total discharge.

From the above field measurements it is concluded that Malpaso, Dike No. 2 has performed satisfactorily, except for the deep seated movements detected in the foundation which may reflect ground adjustments along the fault.



a) Maximum horizontal displacements in the inclinometers and the surface monuments

Inclinometer	Parallel to the axis ↑ (+) ↓ (−)	Normal to the axis (−) → (+)
I	−0.77	+0.49
II	+0.72	−1.09
III	−2.06	−7.40
IV	−2.58	−2.45
V	−4.86	−5.80
V-b	−1.60	−1.09
VI	−2.03	−3.10
VII	−1.55	−0.62
VIII	−0.65	+0.55

b) Variation of horizontal displacements with time, normal to the axis

Curve	Date	Pool level
Origin	Oct. 1964	—
1	May. 1966	86
2	Jan. 1967	177
3	Apr. 1969	164

c) Variation of settlements with time

Origin	Oct. 1964	—
1	Jun. 1966	86
2	Jan. 1967	177
3	Aug. 1967	166.5

d) Piezometric levels, discharge in the drainage gallery, pool elevations and rainfall, july to december 1968

FIG. 7 FIELD MEASUREMENTS AT DIKE No 2, MALPASO

Président A. MAYER

Je remercie vivement le Professeur Marsal pour cette très intéressante communication et maintenant je voudrais donner la parole a Monsieur Pinkerton qui est Ingénieur chargé de la partie Génie Civil des projets de la Société Snowy Mountain en Australie. Vous connaissez l'importance de ces projets, par conséquent l'importance du rôle de Monsieur Pinkerton. Il va nous parler des critères d'éxécution et de contrôle des barrages en enrochement La parole est à Monsieur Pinkerton.

Panelist I. L. PINKERTON (Australia)

SYNOPSIS

The more general acceptance of the technique of placing rockfill in layers and compacting with a heavy flat vibrating roller has led to an increasing use of marginal quality rock compared with the hard crystalline rock required for rockfill placed by high dumping. This trend has highlighted the need for a more positive determination of the quality of rockfill. Various laboratory and field tests are discussed in relation to their suitability to determine the properties of rockfill such as breakdown during winning, handling, placing and compacting the rockfill shear strength of rockfill, movements in rock fill and breakdown of rock fragments after placement, durability of rockfill and quality control during construction.

INTRODUCTION Over the past decade rockfill dams have become increasingly popular and are now often constructed on sites where previously concrete dams would have been built. This trend is universal and is probably due to two major reasons, namely,

(a) the lower unit costs for winning, handling and placing rockfill,

and

(b) the fact that geological weaknesses in the foundations disclosed during investigation and construction are far less likely to cause expensive treatment or design changes than for concrete dams.

The more general acceptance of the technique of placing rockfill in layers and compacting with a heavy flat vibrating roller has resulted in fewer dams being constructed by "high-dumping" the rockfill. With the placing of rockfill in layers there is an increasing usage of marginal quality rock compared with the hard crystalline rock generally required for high dumping. This trend has highlighted the need for a more positive determination of the quality of rockfill and control during construction.

There is not at present a standard series of tests to establish whether a particular rock source would yield acceptable quality rockfill. However, the suitability of rockfill for a particular project may usually be assessed from physical properties determined from both field and laboratory tests. These tests should be sufficiently comprehensive to indicate -

the degree and nature of mechanical breakdown of the rock expected to occur during winning, handling, placing and compacting,

the shear strength of the rockfill,

the anticipated mechanical breakdown of the rock fragments and expected movements under the stresses existing within the dam both during construction and initial reservoir filling,

and

the durability of the rock fragments under the conditions pertaining within the dam.

BREAKDOWN OF ROCK DURING WINNING, HANDLING, PLACING AND COMPACTING The degree and nature of the mechanical breakdown of the rock which would be expected to occur during winning, handling, placing and compacting, may often be assessed from the geological information obtained from a field exploration program, supplemented by laboratory tests. The degree of breakdown depends on the joint pattern in the rock and the nature of the rock itself. Weak or friable rocks such as shales. sandstones, mudstones and schists are examples of rock which may be particularly susceptible to mechanical breakdown.

There are no suitable laboratory tests to simulate mechanical breakdown during winning, handling, placing and compacting. but the general physical properties of the rock gives a fairly reliable indication of the nature of rockfill from a particular rock source. Many laboratory tests used for determining the properties of rockfill are those established for testing concrete aggregate and the criteria used for acceptance are generally not directly applicable to rockfill. Laboratory tests which are often performed include the following -

Unconfined compressive strength of the rock which is usually obtained from tests on samples of drill cores. Low compressive strengths give a general indication that the rock might be subject to mechanical breakdown. Some types of rock exhibit a marked lower compressive strength in the wet condition than in the dry condition and rock samples should be tested both dry and saturated. If an appreciable difference in the compressive strength is obtained, it is advisable to wet the rock during placement in the dam so as to ensure that the maximum breakdown of the rock fragments is obtained during the construction stage. Settlement of the rockfill after construction is completed would then be kept to a minimum.

Los Angeles abrasion which is frequently performed to determine the abrasion resistance of the rock fragments. Although the test is primarily for determining the suitability of rock for concrete aggregate, it provides a useful indication of the ability of the rock fragments to resist mechanical breakdown during handling and placing in the dam.

Aggregate crushing which is sometimes made to determine the crushing resistance of the rock fragments. Saturated rock may give considerably higher crushing values than those obtained from dry samples. This test is also primarily for testing rock for suitability as concrete aggregate but the results give a general indication of the breakdown to be expected in the rockfill during handling and compaction.

Where the rockfill is of marginal quality, however, the only reliable method of determining the amount and nature of the breakdown of the rock that will occur during winning, handling, placing and compacting the rockfill is by means of a trial blast and test fill. The extent of the breakdown may vary depending on the methods chosen for winning, handling, placing and compacting the rockfill and care must be exercised to ensure that the same methods and equipment are used for the test fill as proposed for the actual construction. If the rock exhibits an appreciable reduction in strength when saturated, the test fill should be watered during placing and compacting. Tests should be made to determine the physical properties of the various layers of rockfill and should include gradation, density, permeability, settlement and porosity. If compaction is proposed by means of a roller, tests should be made for varying numbers of roller passes.

Recent specifications for rockfill show a greater tolerance to the inclusion of fine material. However, the amount of fine material which can be tolerated in rockfill will depend on many factors including the maximum particle size, gradation, porosity and the nature of the fine material. Rockfill with a large maximum particle size and high porosity will obviously be able to accept more fine material without losing its rockfill properties than one with a smaller maximum particle size and low porosity. The amount of fine material which could be tolerated in the rockfill may sometimes be dependent on the nature of the fine material. If the fine material is of a granular nature large amounts may well be acceptable but if it contains an appreciable proportion of material passing a 200 mesh sieve, it is more likely to cause the rockfill to lose its characteristic free-draining rockfill properties. For this reason the nature of the fine materials resulting from rock breakdown should be carefully determined during the investigation stage.

Some specifications for rockfill limit the amount of material passing a 1 inch sieve. This limit is often specified up to 30 per cent on the basis that this amount would still ensure adequate permeability or free-draining properties of the rockfill. While this may be a good general figure to specify it would be more satisfactory to determine the limiting percentage of fine material from gradation and permeability tests made on the test fill. An indication of how much this limit varies might be obtained from two rockfill dams, one recently constructed and one under construction in the Snowy Mountains of Australia -

At Blowering Dam which was completed in 1968 the rockfill consisted of quartzite siltstone and phyllite with a maximum size of about 18 inches. Tests made on the fill during construction showed that 80 per cent of the tests gave 18 to 27 per cent of the rockfill smaller than 1 inch. A few tests in the phyllite gave up to 50 per cent smaller than 1 inch but adequate permeability was still achieved.

At Talbingo Dam (Ref. 1), the rockfill is rhyolite. The dam is under construction and tests to date show that the percentage of material smaller than 1 inch varies from 20 per cent for rockfill with a maximum size of 36 inches, to 54 per cent for rockfill with a maximum size of 6 inches. The finer fractions of the rockfill are, however, of a granular nature and adequate permeability or free-draining properties are obtained.

SHEAR STRENGTH OF THE ROCKFILL Triaxial tests are often performed, particularly for major dams, to obtain the shear strength of the rockfill for use in embankment stability calculations and also to obtain the stress-strain relationship. The tests should cover the range of stresses expected in the completed dam. It is usual for the gradation of the test specimens to be a scaled reduction of the expected actual gradation of the rockfill, with the maximum particle size equal to one-fifth of the diameter of the test specimen, although when the test is performed on a large capacity triaxial machine, the test specimen may sometimes consist of the smaller rock sizes of the expected gradation of the rockfill. In the case of rock with an appreciably lower strength in the saturated condition, the triaxial tests should be made with the test specimen in the wet as well as the dry condition. The stress-strain relationship obtained from triaxial tests provides basic data for calculating the anticipated movements which would be expected to occur in the rockfill zones of the dam and the gradation curves made before and after testing will give an indication of the expected breakdown of rock fragments under stress.

BREAKDOWN OF ROCK FRAGMENTS AND MOVEMENTS IN ROCKFILL UNDER STRESS Particle breakdown may occur under stress either by cleavage along incipient joint planes or by fracture of the rock itself. This particle breakdown, together with crushing of the sharp edges of the rock fragments results in movements within the rockfill under stress and may sometimes cause some loss in permeability. The data obtained from

Fig. 1(a). Blowering Dam. Phyllite after two passes of vibrating roller.

Fig. 1(b). Blowering Dam. Test pit excavated in phyllite.

triaxial tests provides basic information for determining the extent of particle breakdown and movements likely to occur in rockfill under stress. In some dams large movements of the rockfill may critically affect the safety of the dam, as in some cases of high rockfill dams with upstream membranes of concrete, or dams with thin non-plastic cores where large movements of the supporting rockfill may cause cracking of the core.

In many types of dams quite large movements of the rockfill are acceptable without affecting the ability of the dam to fulfil its function. An illustration of this is Blowering Dam where fairly large movements have occurred in the rockfill, both during the construction and initial filling of the reservoir. Blowering Dam is an earth and rockfill structure with a central core composed of a soil of medium plasticity. The width of the core is approximately equal to the hydraulic head. The dam is 376 feet high and the outer slopes of the rockfill are 1.75 to 1 in the top half and 2 to 1 in the lower half. The rockfill consisted of hard quartzite and siltstone which was mostly placed in the inner zones of the rockfill, and soft phyllite (Fig. 1) grading into siltstone which was mostly placed in the outer portion of the rockfill zones. All the rockfill was placed in layers and compacted with a heavy vibrating roller.
In the inner zones of the rockfill the layer thickness was 3 feet while in the outer zones it was 6 feet. The horizontal movements which occurred in the downstream rockfill zone were a maximum at about 120 feet up from the foundations and varied between 2.8 feet adjacent to the core to 2.2 feet at the downstream surface Fig. 2.

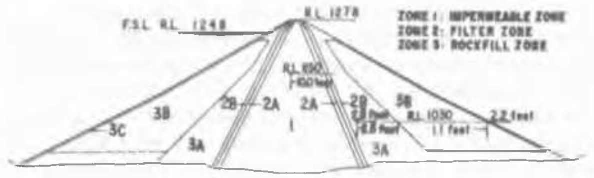


Fig. 2. Blowering Dam. Showing total horizontal movement of downstream rockfill and settlement of core recorded during construction and initial filling.

Eighty per cent of this movement took place during construction and the remaining 20 per cent over a period of 6 months during which the reservoir was filling. As a result of the large horizontal movements which occurred in the rockfill, much larger vertical settlements of the central core were recorded. The maximum settlement occurred 120 feet below the crest and was of the order of 10 feet.

It is apparent that large movements may occur in soft, low-strength rock before the rockfill mobilises its shear strength and where large movements are expected to occur, this factor must be taken into consideration in considering the acceptability of rockfill for a particular project.

The horizontal movements and vertical settlements recorded at Blowering Dam are very much greater than those recorded at Geehi Dam which is also a rockfill dam with a central core. Geehi Dam which was completed in 1966 (Ref. 2), is 300 feet high and has a core width approximately equal to half the hydraulic head. The downstream outer slope of the rockfill is 1.75 to 1 while the upstream rockfill slope is 1.75 to 1 in the upper section and 2 to 1 in the lower section. The rockfill consists of granitic gneiss which is hard, durable and massive (Fig. 3).

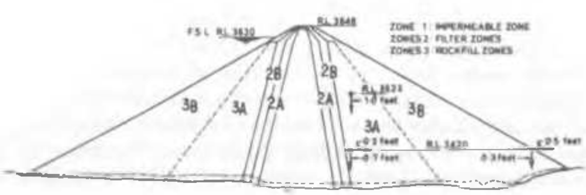
The inner zones of the rockfill were placed in 3 feet layers and compacted with a heavy vibrating roller while the outer zones of the rockfill which contained the larger rock sizes, were dumped in lifts ranging from 90 feet in the lower levels to 10 feet in the upper levels. The horizontal movements measured in the downstream rockfill zone at about 100 feet up from the foundations were 0.2 feet adjacent to the core and 0.5 feet at the downstream face (Fig. 4).

Practically the whole of these horizontal movements took place during construction. Maximum vertical settlement of the rockfill occurred adjacent to the core and about 200 feet up from the foundations and amounted to 1.0 feet. Almost the whole of this settlement occurred during construction.

DURABILITY OF ROCK FRAGMENTS. The durability of rock fragments under conditions pertaining in the dam will depend largely on whether the rock contains minerals subject to breakdown or volume change, or has other physical weaknesses which would cause the rock to disintegrate progressively. A thorough petrological examination of the rock should always be made but engineering judgment still plays a large part in assessing the suitability of rock in this respect. Inspection of existing dams constructed of marginal quality rock often gives valuable information on the degree of deterioration in this respect.

Rockfill contained within an embankment is subject to fairly constant conditions of temperature and moisture although some sections of the upstream zones are alternately wetted and dried with reservoir fluctuations. Only the surface rock is exposed to severe climatic conditions with daily temperature cycles and possibly freezing and thawing during the winter months. Some types of rock which would be subject to rapid breakdown at the surface may remain intact

Fig. 3. Geehi Dam. Placing of rockfill in layers.



4. Geehi Dam. Showing total horizontal movements and vertical settlements of downstream rockfill recorded during construction and initial filling

under the constant conditions within an embankment where there is little or no air circulation.

Accelerated weathering tests such as sodium sulphate soundness are often of considerable value but may sometimes prove to be too severe and give misleading results. Sodium sulphate soundness tests on the phyllite and siltstone rock used for Blowering Dam produced losses ranging from 21 to 35 per cent which are very much higher than the 10 per cent limit frequently specified for concrete aggregate acceptance. This phyllite and siltstone rock gave quite satisfactory rockfill and it would appear that, while the sodium sulphate test is a useful guide to the durability of the rockfill, a lower standard of acceptance to that usually adopted for concrete aggregate, would be satisfactory. Freezing and thawing tests are also useful in assessing the durability of the rock fragments and ability to resist mechanical breakdown.

Wetting and drying tests are considered to be more satisfactory than the accelerated weathering tests, and are less likely to lead to rejection of marginal quality rock which would make satisfactory rockfill. This type of test has been used by the Snowy Mountains Authority, the procedure being to subject a sample of different particle sizes to alternate wetting and drying over an extended period. The cycle consists of soaking the sample in water for 21 hours, oven-drying at 50°C for 24 hours and air-cooling for 3 hours. After each cycle the increase in breakdown of material passing a particular sieve size is recorded. The aim of the test is to determine the rate and nature of the breakdown of the rock fragments over an extended period.

QUALITY CONTROL DURING CONSTRUCTION The acceptability of rockfill with respect to quality and permissible fines is usually determined by inspector judgment at the quarry face. The general appearance of the quarried rock and the extent of breakdown which occurs during the quarry operations are the main factors in the determination of acceptability. Particular attention needs to be given to the sections of the quarry where either deep weathering or shear zones occur. The inspector is guided in his judgment by the results of laboratory data and test fills and, of course, by the nature of the rockfill after placing and compacting in the dam.

The control of the quality of rockfill is now greatly assisted by regular sampling and testing for gradation and density. The placing of rockfill in layers and compacting with a flat vibrating roller has considerably facilitated the sampling and density testing procedure which is usually carried out by the now well-known water replacement method using a plastic sheet and large test ring which lies conveniently on the flat, compacted surface of the rockfill layer. The test rings are usually 6 feet in diameter, although occasionally 8 feet diameter test rings are used. The rockfill from these tests has a considerable volume and may be used for large size triaxial tests to ensure that the strength parameters assumed in the design are being achieved in the dam.

During the dumping of rockfill in the dam some segregation of the rock particles will inevitably occur. When the rock is placed by "high dumping", this segregation is very pronounced and it is usually necessary to resort to sluicing to disperse the fine material which remains at the top of each lift. Sluicing should be sufficient to wash the fine material into the void space between the larger rock fragments. When the rockfill is placed in layers, the amount of segregation varies depending on the thickness of the layer. For a 3 feet layer thickness no significant segregation normally occurs but for a 6 and 9 feet layer thickness, the amount of segregation may become quite pronounced. Dispersion of the fine material, however, is not normally necessary for rockfill placed in layers, unless the segregation and nature of the fine material is likely to result in extensive areas of low strength material.

Wetting the rockfill during dumping and placing, which is usually specified for rockfill exhibiting appreciably lower strengths in the saturated condition, is sometimes helpful in indicating whether marginal quality rock is "free-draining".

CONCLUSIONS

For all major dams and wherever rockfill is considered to be of marginal quality, extensive testing should be made to determine the physical properties of the rockfill.

The degree of mechanical breakdown of the rock fragments during winning, handling, placing and compacting of the rockfill may be assessed from the physical properties as determined from laboratory tests. However, there is no suitable laboratory test to simulate this breakdown and a test fill is often the only reliable means of establishing the nature and performance of the rockfill for a particular project.

The anticipated mechanical breakdown of the rock fragments under stress and the stress-strain relationship of the rockfill, should be determined from large size triaxial tests so that expected settlements in the dam may be determined. This may be particularly important where soft, low-strength rock is under consideration or where cleavage along incipient joint planes may cause large movements of the rockfill during construction of the dam and initial filling of the reservoir.

The durability of the rock fragments should be assessed from their mineral composition, supplemented with accelerated weathering tests. Wherever possible examination of older rockfills constructed of similar rock would be an advantage. However, the criteria for acceptance for concrete aggregate frequently used in the accelerated weathering tests are considered to be too severe and may cause rejection of rock which would be satisfactory as rockfill. Some relaxation of these criteria is warranted, particularly as the rockfill in embankment dams is under fairly constant conditions and not exposed to the severity of the weather.

Control of rockfill on the embankment is generally limited to visual methods, based on the results of laboratory tests and field trials. Regular gradation and density tests should be made on the fill and the spoil from these tests may be used for large triaxial tests. Sluicing is necessary to disperse the fine material when the rockfill is placed by "high dumping". However, when the rockfill is placed in layers the amount of segregation is usually not sufficient to affect the quality of the rockfill.

REFERENCES

JOHNSON, K.E., AUBREY, K.G. and MACPHERSON, P. M. - Tumut 3 Project - Concept and Investigations Jour. I.E. Aust., Vol. 39, No. 7-8 pp. 123-137.

PINKERTON, I.L. and PATON, R.J. - Design and Construction of Geehi Dam. Jour. I.E. Aust., Vol. 40, No. 3 pp. 33-48.

Président A. MAYER

Merci, Monsieur Pinkerton, pour vos très intéressantes observations, et maintenant je voudrais donner la parole au Docteur Nichiporovich. Celui-ci est Directeur de l'Institut Fédéral de recherches de l'U.R.S.S. il va nous parler de la relation entre les contraintes et des déformations des noyaux en argile des barrages.

Panelist A. A. NITCHIPOROVITCH (U. S. S. R.)

It is known that earth and rockfill dams are undergoing complicated deformations (vertical, normal, transverse and longitudinal) in three-dimensional system during construction and operation periods.

The most significant and intense deformations occur during the construction period. The shape and steepness of canyon walls affect essentially the value and direction of all deformations. It is particularly obvious when the longitudinal section of a dam core is considered.

Longitudinal deformations of such dams as Serre-Ponçon Dam, Peruca Dam, Tooma Dam, RectorCreek Dam were observed by surface bench marks while on El Infiernillo Dam, Gepatsch Dam, Mammoth Pool Dam, Netzahualcoytl Dam, Muddy Run Dam, Plover Cove Dam, La Villita Dam and other dams these deformations were observed with measuring devices embedded in

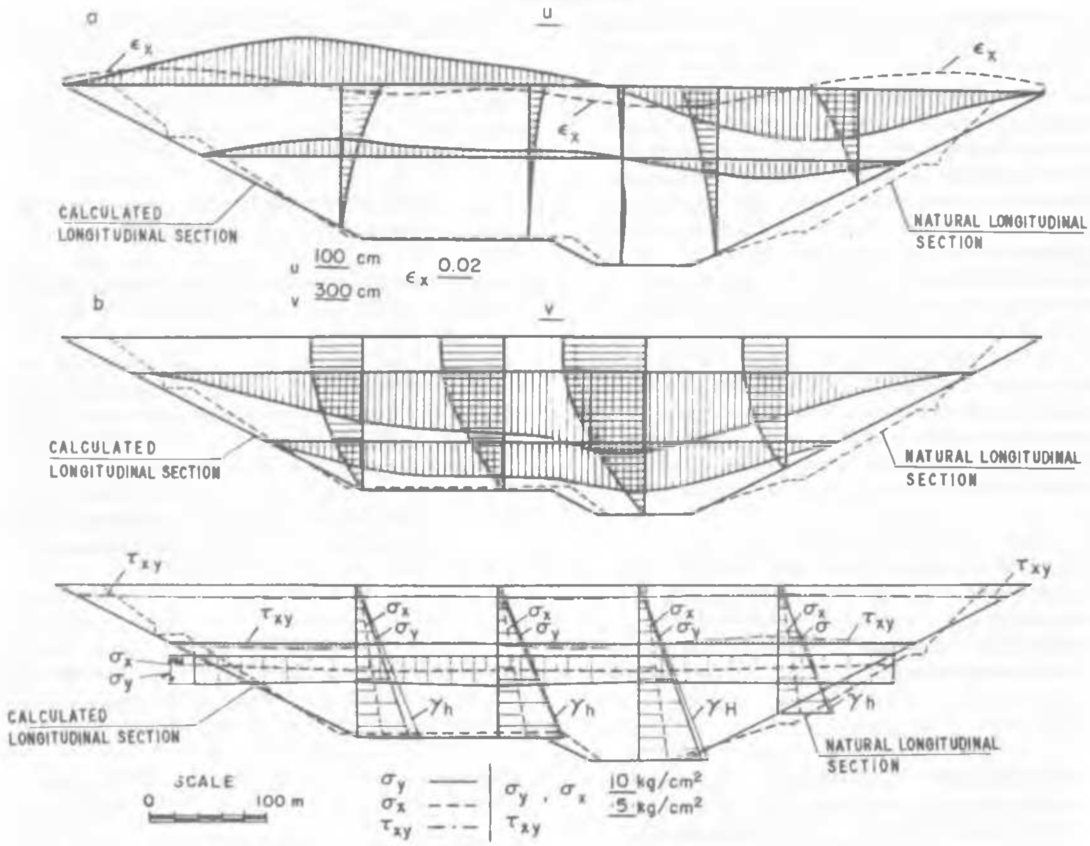


Fig.I
The results of the analysis for a dam of $H_{max.}$= I50m.

a)Vertical deformation curve V
b)Horisontal displacement U and curve of crest strains ε_x
c)Vertical σ_y, horisontal σ_x, and tangential τ_{xy} stresses

dam cores.

These measurements reveal that both compressive and tensile deformations occur through axis of a core. The latter, as a rule caused cracking developing from the crest surface and extending sufficiently deeply into the dam foundation.

Up to date the analysis of these deformations are not available yet. In I960-I963 Leonards and Narain (USA) proposed a solution to the problem.

They showed the problem for a two-dimensional case, considering a cross-section of the dam along its axis as a beam fixed at the abutments, using the theory of elasticity. The tensile deformations can be determined from the equation, developed by them, e.i. $\varepsilon_x = f(L,H)$, where ε_x - the longitudinal strain, H and L - the height and crest lengh of a dam respectively.

By the comparison of computed and laboraratory results for any core material the problem of cracking of a core can be studied. But this solution is considered adequate only for relatively low dams, when the crest lengh - height ratio ($\frac{L}{H}$) is significant and for dams which settlements are observed but not computed.

For dams with $\frac{L}{H}$ = 0,5 - 2,0 and of a significant heigh with steep abutments the problem of core cracking is not solved yet.

In the USSR the problem of cracking in dam cores is intensely studying as many high earth and rockfill dams are under construction. In particular, the methods of deformation analysis of earth and rockfill dams are developed based on the theory of elasticity (for one-dimensional system) using method of finite differences of elements and electric computors. This analysis is used both for the transverse (the core deformations, unloading of a core with various density of shell materials) and longitudinal directions.

This paper deals only with the longitudinal section of a core.

As an example, the solution of the problem is given for a high dam. The analysis of dam displacements is considered for two-dimensional system of the theory of elasticity, neglecting tangential stresses along the contact of a core with the shell prizms.

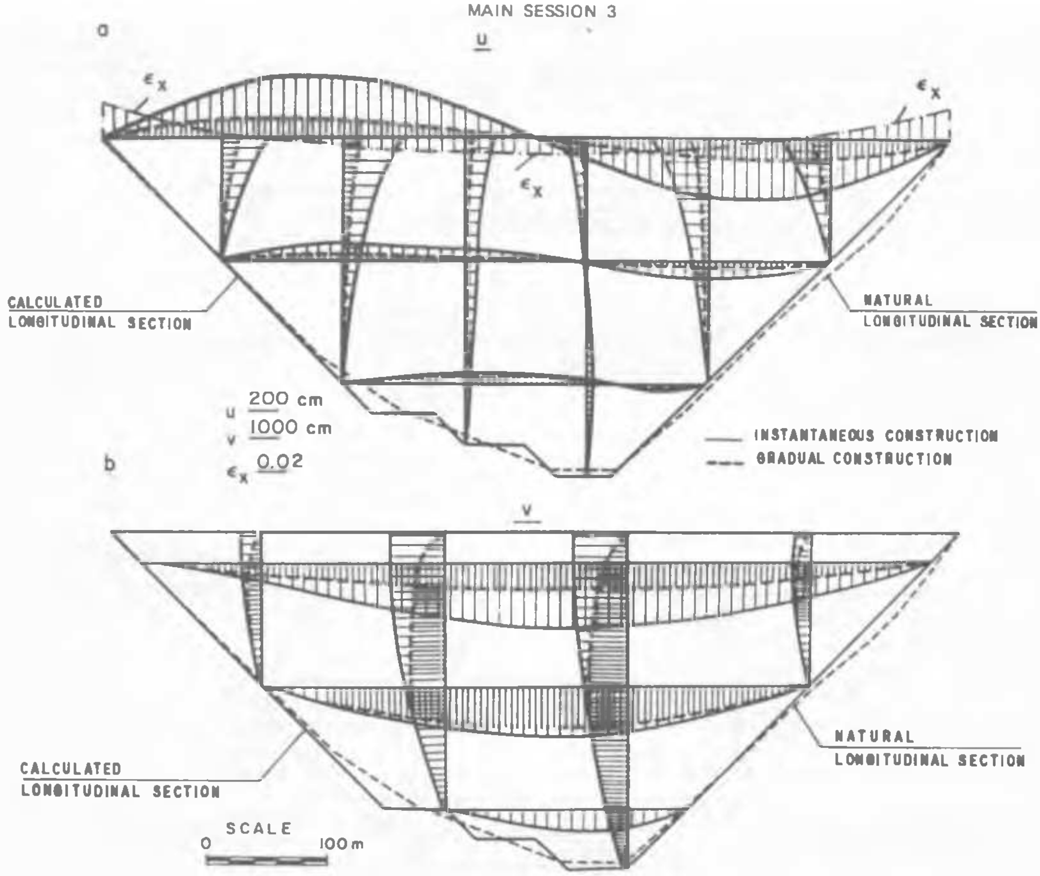


Fig.2

The curves of horisontal displacements U,V and srains ε for the dam, H_{max}= 300m.

a) -----instant filling

——gradual filling

b)Vertical displacements

-----instant filling

——gradual filling

Consideration of two limiting two-dimensional cases - one with no lateral expansion of the core in the transverse direction (maximum lateral pressure of adjacent shell prizms) and the other with unlimited lateral expansion of the core (without lateral pressure of shells enables the encorporation of certain corrections in the results.

The canyon is of assimetrical shape; modulus of deformation E =I50 kg/sq.cm., Poisson's ratio μ = 0,32; unit weight of material γ_{ck}=2,0 t/cu. m.

The solution is developed in terms of displacements using Lame's equation.

The system of equations is solved using Zeidel's method by БЭСМ-2 computor. For approximation of the considered area - the number of joints of the lattice was assumed as I86 and 89. The lattice for the determination of U was displaced in relation to the lattice for V by I/2 spacing of lattice. (U and V are vertical and horisontal displacements).

These displacements were neglected along the contact of the dam crest and canyon walls*. For this solution the construction period without upstream pressure of water was considered. All determination were made for the cases of instant and gradual filling of a dam.

For the case of gradual filling of a dam the number of layers (four layers in our example) were assumed and the solution is given for each layer assuming that the deformation of the considered layer is imposed only by the load above. Consequently, when the dam is filled gradually deformations and stresses are minimum and stress-strain distribution differs from the distribution for the instantly-filled dam.

* This is true only for the cases, when the cohesion force along the contact area is not more than the shearing resistance of core materials.

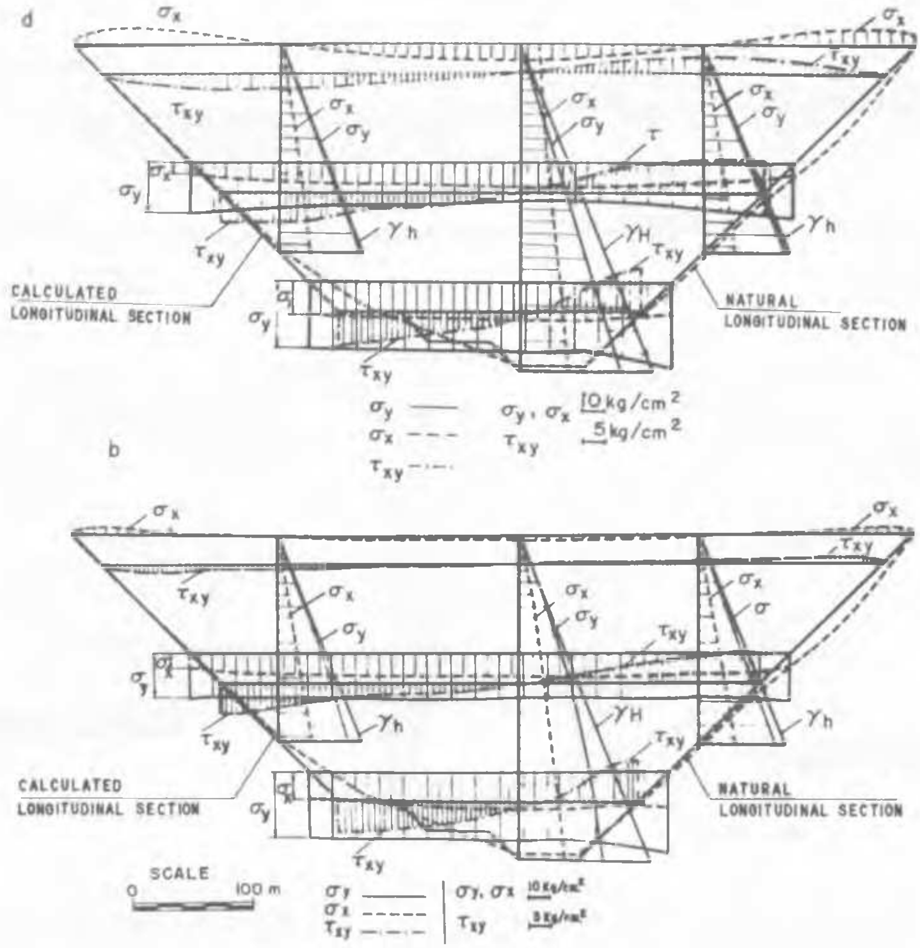


Fig.3
The results of stress analysis for the dam of H_{max}= 300 m.

a)stresses $\sigma_x, \sigma_y, \tau_{xy}$ for instant filling
b)Stresses $\sigma_x, \sigma_y, \tau_{xy}$ for gradual filling

The results of the analysis for the dam of H_{max}= 150 m. are shown in fig.I.

The vertical deformation curve is shown in Fig.Ia. The horizontal displacements and curve of crest strains ε_x are plotted in Fig.Ib. The vertical (σ_y) and horizontal (σ_x) stresses are plotted in fig Ic. All these values are determined when the dam is filled instantly. The results of the analysis for the dam of H_{max}= 300 m. are shown in fig.2. The curves of horizontal displacements (U) and strains ε_x for instant and gradual filling are shown in fig.2a. The curve of vertical displacements (V) for instant and gradual filling is shown in fig. 2b.

The results of stress analysis (σ_x, σ_y, τ_{xy}) for instant (3a) and gradual (3b) filling are plotted in fig.3.

From the above, the following conclusion are drawn:

The results of the analysis for instant and gradual filling differ greatly. For instant filling all the values very much higher (2-3 times) than those for gradual filling. The lattice method can be used for the analysis of the longitudinal section of a dam in the canyon with the complicated assymetrical shape using the theory of elasticity.

Using this analysis the values of core strains can be determined as well as the areas of tensile stresses and cracking can be predicted.

In particular, for the considered dams the areas of cracking in cores were predicted while the dam materials are undergoing strains up to 0.02 without failure.

The deformation distribution was confirmed by the observations of the latest dams as on account of crest strains as arch effect in the longitudinal direction. In the first case (the wide canyon) the arch effect is not obtained in practice while in the second case (the narrow canyon)

σ_y is~35% less than σ_H

For the similar canyon these results can be used for estimation of deformations and stresses of a core, using the following relations for the modelling:

$$\sigma = \bar{\sigma}\frac{H}{\bar{H}}; \quad \varepsilon = \bar{\varepsilon}\frac{H}{\bar{H}}\cdot\frac{\bar{E}}{E};$$

$$u = \bar{u}\left(\frac{H}{\bar{H}}\right)^2; \quad V = \bar{V}\left(\frac{H}{\bar{H}}\right)^2\cdot\frac{\bar{E}}{E}$$

where $\sigma, u, V, \varepsilon, E$ - stresses, displacements and deformations, dam height, modulus of elasticity for the dams of any height and $\bar{\sigma}, \bar{u}, \bar{V}, \bar{\varepsilon}, \bar{E}$ - the values, shown in figures I,2,3. In the future all the results of similar analysis can be defined more precisely by varying the initial data E and μ depending on the stresses at individual points, taking into consideration the three-dimentional system and analysing simultaneously all characteristics along sections normal to the ones considered and so on.

Président A. MAYER

Je remercie vivement le Docteur Nichiporovich pour les indications rapides qu'il a bien voulu nous donner sur les études en cours pour le barrage de 300 m qui est, je crois, en construction, en U.R.S.S. J'espère qu'à l'occasion du prochain Congrès nous aurons la possibilité de nous rendre compte de l'avancement non seulement des études mais des travaux de cet ouvrages qui bat, je crois, de loin, tous les records du monde au point de vue de la hauteur.

Et maintenant je vais donner la parole à Monsieur l'ingénieur Sembenelli du Bureau Electro-Consult d'Italie qui va nous parler d'une question extrêmement intéressante: L'utilisation des revetêments métalliques dans les barrages en enrochement.

Panelist P. SEMBENELLI (Italy)

The State of the Art report has made no mention of dams with impervious membrane on the upstream slope. Far from pretending to treat this subject as it was done in the State of the Art, I wish to add a few remarks.

To remain within discussion limits, I will speak of impervious membranes made of a metal sheet. Out of 38 dams on records, whose watertightness is ensured by a metal membrane, 14 are of rockfill type with metal plate on the upstream slope.

Dams up to 63 m in height (Salazar)have been successfully built, and dams over 60 years old (Skagway) are still in satisfactory shape.

One may ask why then, has this solution been adopted only in a relatively limited number of cases?

I believe that fears of oxidation prevented, in most instances, the use of metal in connection with dams. But experience gained so far has been surprisingly favorable and disproving.

Metal can be of different chemical composition. I could find nine "pure iron" facings (Aguada Blanca), three mild steel facings (Sirinumu), seven carbon steel facings (Lago Verde), seven copper steel facings (Salazar). In all cases, the metal facing behaved satisfactorily.

Protection of metal surface can differ widely. Epoxy resin films (Alpe Gera), lead oxide paints (El Vado), bituminous paints (Salazar), bitumen coating(Bever), asbestos bond (Heart), and nothing at all (Lago della Vaca) have been used, as well as cathodic protection (Lagartijo). Results have been equally good.

Maintenance is sometimes reported at 15 years intervals and sometimes as none (Catamount Cr.). In most cases what is done yearly is only inspection.

Corrosion records are positive irrespectively of dam elevation and environment. Measurements carried out on metal facings in Italy over a decade, showed a surprisingly limited metal loss (4 g/100 cm^2, or 50 microns for copper steel). Carbon steel shows an over-

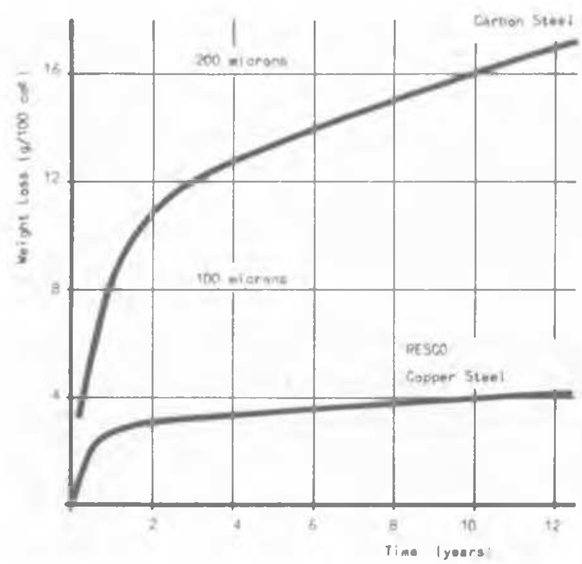


Fig. 1. Corrosion of metal facings

all loss four times greater but mostly taking place in the first years of operation.

NAME	YEAR of metal facing placement	COUNTRY	RIVER	CREST EL. m a.s.l.	DAM HEIGHT m	FACING AREA m²	METAL	THICKNESS mm	WEIGHT kg/cm² overall	SEAMS m spacing of weldings	JOINTS m spacing	SLOPE hor/1 vert	BEDDING m of thickness m horizontal	COATING WATERSIDE number correspond to coats	COATING DRYSIDE number correspond to coats	MAINTENANCE frequency in years	SETTLEMENTS cm
1 METAL PLATE ON UPSTREAM SLOPE OF EARTH AND ROCK DAMS																	
1 - 1 Rock Fills																	
AGUADA BLANCA	u. c.	Peru	Chili	4 171	45	3 500	iron (n)	5	44	2.5 x 4	10 vert 16 hor (u)	1.7	crushed rock 2 + crushed rock 3	epoxy resin 2	epoxy resin 1	—	—
LAGO VERDE	1966	Italy	—	2 531	48	18 000	Carbon steel	4	—	1.6 x 4	8 vert 8 hor (u)	.9	concrete .4 + rubble work 2 to 4	sand blast + red lead + epoxy resin 2	none (u)	none	2 vert 1.5 hor
SIRINUMU	1962 (c)	N. Guinea	Laloki	534	23.5	3 700	mild steel (m)	9.5 (s) / 6.3	—	—	9 vert	1.3	crushed rock 1 (1.5" max.)	coal tar epoxy resin 2 + cathodic protection	coal tar epoxy resin 2	—	—
LAGARTIJO	1958	Venezuela	Lagartijo	175	27	—	Carbon steel	6.3	—	1.2 x 2.5	10.5 vert	1.5	grits and sand	cathodic protection	none	none	—
VIEILLES FORGES	1949	France		245	10	1 000	Copper steel (VIRESCO) (l)	4.	35	1.5 x 6	1.5 vert	.4	rubble work 1 to 2	—	—	—	—
KHRAM (a)	1948	U.R.S.S.	Khrami	1 500	35	6 800	Copper steel (l)	6.	42	2.5 x 6.1	12 vert	1.35	concrete + rubble work 3 to 5	—	—	—	18
SALAZAR	1948	Portugal	S. Catarina	56	63	7 000	Copper steel (l)	8 / 6.3 (r)	84	2.5 x 7.5	7.5 vert	1.25	concrete .3 + masonry 1 to 3 + rubble work	priming cutback + + waterpr. flinthote + + wild paint	bituminous flint-kote + waterpr. flintkote 2	10	4 vert 35 hor
SOCI (?)	1936 (?)	U.R.S.S.		99	43	11 000	—	6.	—	2.5 x 6.1	12 vert	1.5	concrete + masonry	—	—	—	—
EL VADO (a)	1934	USA (N.Mex.)	Chama	2 109	53	18 000	—	6.3	67	2.5 x 9.1	—	2	gravel	red lead + metal paint	none	—	—
PENROSE (aa)	1932	USA (Colo.)	E. Beaver Cr.	2 950	30.5	—	Copper steel (l)	9.5 / 6.3 (r)	—	2.5 x 6.2	12.5 vert	.5	masonry + rubble work 0.6 to 2.8	bituminous paint	—	—	—
BEAVER PARK (bb)	1910	USA (Colo.)	Beaver Cr.		36	—	—	—	—	—	—	—	—	—	—	—	—
CHEESMAN (?)	1904	USA (Colo.)				—	—	—	—	—	—	—	—	—	—	—	—
EAST CANYON	1902 (b)	USA (Utah)			36	—	—	—	—	—	1.8 1 (r)	—	—	—	—	—	—
SKAGUAY	1900	USA (Colo.)	Beaver Cr.		23	—	—	12.5 / 9.5 (r)	—	—	—	.6	—	bituminous paint	none	15	—
1 2 Earth Fills																	
RUHRTAL	1949	Germany	Ruhr		59	—	—	—	—	—	—	2.75	—	—	—	—	—
S. CATAMOUNT CR.	1936	USA (Colo.)	S. Catamount	2 814	34 (i)	14 000	Copper steel (l)	6.3	—	2.5 x 7.5	—	2	sand	red lead + metal paint	—	5 + 10 (z)	—
CRYSTAL CR.	1935	USA (Colo.)	Chrystal			—	Copper steel (l)	6.3	—	—	—	—	sand	red lead + metal paint	—	5 + 10 (z)	—
PIKES CR.	1912	USA (Pa.)	Pikes Cr.		18	—	—	—	—	—	—	—	—	—	—	—	—
2 METAL PLATE INSIDE EARTH AND ROCK DAMS																	
2 1 Rock Fills																	
LOWER OTAY	1897 (e)	USA			40	—	Carbon steel	8.6 / 6.3	—	—	—	vert	—	concrete	concrete	—	—
EAST CANYON	1896 (d)	USA (Utah)	East Canyon Cr.		20												
2 2 Sand, Earth, Zoned Fills																	
HEART	1954	USA (Colo.)		35	23.	—	Carbon steel	—	—	—	—	vert	—	asbestos bounded	asbestos bounded	—	—
RYBINSK	1941	U.R.S.S.	Volga		20~	—	Carbon steel (q)	—	—	—	—	vert	—	—	—	—	—
UGLIC	1940	U.R.S.S.	Volga	30	22~	6 000	Mild steel (p)	14 / 11	—	—	—	vert	—	—	—	—	—
BEVER	1938	Germany	Wupper	300	23~	—	Mild steel (p)	8	—	2 x 2	—	vert	—	bitumen	bitumen	—	—
IVAN'KOVO	1937	U.R.S.S.	Volga	—	41	—	Carbon steel (a)	—	—	—	—	vert	—	—	—	—	—
3 METAL PLATE ON UPSTREAM SLOPE OR MASONRY AND CONCRETE DAMS																	
3 1 Concrete Blocks Dams																	
EL FRAYLE	1960	Peru	Chili		18~	1 400	iron (n)	—	—	3.7 x 4	3.7 vert 4 hor (u)	.65	—	red lead 3	—	—	—
PIAN PALÚ	1959	Italy	Noce	1 802	50	3 000	iron (n)	5/6	—	3.7 x 4	3.7 vert 4 hor (u)	.65	—	—	—	—	—
POZZILLO	1958	Italy	Salso	367	59	9 300	iron (n)	6	—	3.7 x 4	3.7 vert 4 hor (u)	.65	—	—	—	—	—
FANACO	1954	Italy	Platani	681	66	6 500	iron (n)	6	100	3.7 x 4	3.7 vert 4 hor (u)	.65	—	—	—	—	—
3 2 Masonry Gravity																	
LAGO D. VACCA	1949 (g)	Italy	Caffaro	2 360	20.5	800	iron (n)	2.7	—	1.8 x 7	none (v)	vert	—	none	none	—	—
LAGO GABIET	1935 (l)	Italy	Lys	2 378	40	4 800	iron (n)	5	58	2 x 11	none (v)	0.04	—	red lead 2 + aluminum paint	red lead 2	—	—
LAGO D. DIAVOLO	1931	Italy	Brembo	2 145	24	800	iron (n)	2	30	2 x 24	none (v)	vert	—	bituminous paint	none	—	—
3 3 Concrete Gravity																	
ALPE GERA	1966	Italy	Cormor	2 128	175	40 000	iron (n)	3	30	1.4 x 6	6 vert	0.03	—	epoxy resin 1	none	none	—
3 4 Concrete multiple Arcs																	
LAGO VENINA	1965 (h)	Italy	—	1 823	49.5	7 200	Copper steel (RESCO) (l)	3.5	41	1.6 x 2.5 (t)	none	vert	porous concrete	sand blast + epoxy resin 1 + wite paint	none	none	—
4 METAL PLATE ON STEEL FRAMED DAMS																	
HAUSER LAKE	1906 (hh)	USA (Mo.)	—	1 200		—	—	9.4	—	—	—	1.5	—	—	—	—	—
REDRIDGE	1901	USA (Mich.)	Salmon trout	28	23	—	—	—	—	—	—	1	—	—	—	30~	—
ASH FORK	1898	USA (Ariz.)			13	—	—	9.4	—	—	—	1	—	—	—	8	—

(a) gravel fill compacted in 8" lifts
(aa) also P.-Rosemont and E.Beaver Cr.
(b) out of service 1915
(bb) also Goose Neck
(c) to be raised
(d) raised 1900 and 1902
(e) overtopped 1916
(f) built 1922
(g) built 1928
(h) built 1925
(hh) failed 1907
(i) 21.5 m metal plate
(l) typical Copper Steel composition
Cu = 0.28% Mn = 0.47% P = 0.019%
S = 0.02% σ_y = 47 kg/mm² ε_y = 26%
"VIRESCO" contains Cu, Cr, Mo
"RESCO" contains Al, Cr, Cu, P
σ_y = 42÷50 kg/mm² ε_y = 27% min
(n) Typical iron composition
Cu = 0.15% max S = 0.03% max
P = 0.01% max
P + S + C + Mn + Si = 0.1% max
σ_y = 19 kg/mm² ε_y = 38%
(o) upper part wood
(p) standard steel plate
(q) upper part reinforced concrete
(r) upper half
(s) below permanent water level
(t) vacuum box controlled
(u) each joint runs a double span and than is interrupted. Joints are continuour in alternating direction: hor - vert
(v) deformation in horizontal direction permitted by lead caulked joints
(w) monosodic phosphate added to reduce effect of ground currents
(z) parts under minimum water level receive no maintenance

Table 1 Dams with Metal Membranes

In some cases (Lago Venina) metal plates have been the only material capable of withstanding extremely severe climatic conditions (over 100 freezing and towing cycles/year) that proved unbearable to concrete and bituminous facings which had been unsuccessfully tried in several ways for 30 years before metal was adopted.

Metal plate thicknesses from 2 to 12 mm have been used. Recent designs of rockfill type dams, indicate plates 5 to 6 mm thick as appropriate for dams of 60 m height and over. Facings, with just waterproofing role, only 2 mm thick (Lago del Diavolo) are effective since 38 years.

Where vertical joints are not enough to provide the required flexibility, crossing accordion joint systems can be used to take any amount of deformation.

Looking now to other aspects of such dams, thicknesses of 5 to 6 mm allow an overall facing weight of 45 to 50 kg/m^2.

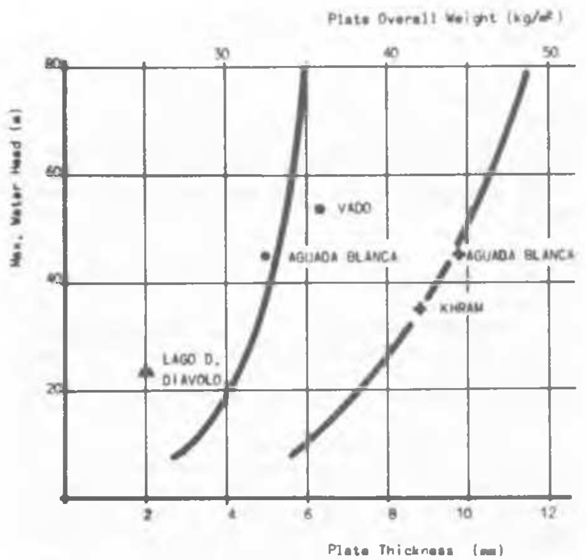


Fig. 2. Thickness and Weight of Metal Facings

Pure iron and copper steel alloys (in place) are rated at 0.5 $/kg with a resulting facing cost of 25$/$m^2$.

Bedding layers, once rubble work, masonry or lean concrete, are now simply made of compacted gravel or crushed rock courses built in the same way as the rest of the embankments.

Metal plates are yard assembled and laid down on bedding. In place welding is so limited to seams 4 to 8 m apart. Up to 200 m^2 of facing per day have been placed at some jobs (Lago Verde).

Embankment upstream slopes can be set at 1.5/1 without special need for hanging the plates to the dam. The downstream slope can be made as steep as one can.

All dams with upstream impervious membranes permit the water thrust to be applied and directed in the best way and, as a consequence, require minimum embankment volume.

Embankment bulk is of a single rock class with large tolerances. This results in maximum construction output, simple placement procedure, reduced need for controls, limited stoppages.

Floods can be passed through and over the special type of rock fills we are speaking about. Recent model and full scale experiences have proved the applicability of such method to advantage in obtaining a sizable limitation in cost of diversion works and construction time.

Président A. MAYER

Je remercie vivement Monsieur Sembenelli pour sa très intéressante communication. Nous sommes au bout maintenant des exposés faits par les Membres du Bureau, mais avant de nous séparer pour 20 minutes, je voudrais vous rappeler les indications que j'ai données au commencement de la séance, alors qu'un certain nombre d'entre vous n'étaient pas dans la salle. J'ai indiqué que nous nous étions mis d'accord pour supprimer toutes les communications verbales par des personnes de la salle, de leur demander de faire leur communication par écrit, et de les faire taper sur les papiers spéciaux dont dispose le Comité Mexicain pour qu'elles puissent être publiées dans le dernier volume des Comptes-Rendus. Par contre, tous les auditeurs sont invités à nous soumettre les questions qu'ils désireraient voir discuter ou sur lesquelles ils voudraient avoir l'avis des Membres du Bureau, et non seulement des Membres du Bureau mais aussi des personnes dans la salle qui désireraient intervenir sur un point ou sur un autre.

A cet effet, je leur demande de bien vouloir, pendant l'interruption de séance, remettre au Secrétaire de séance, Monsieur Moreno, qui est assis tout-à-fait à gauche, du Bureau, c'est-à-dire à votre droite, le texte de la question de façon que nous puissions l'examiner durant l'interruption de séance et tâcher d'en faire un classement logique.

Noue nous empresserons les uns ou les autres d'y répondre ou de susciter des réponses de la part des personnes de l'assistance et comme je vous l'ai dit tout à l'heure, nous espérons que ces modalités donneront un peu plus d'animation et provoqueront une véritable discussion que nous désirons donner à cette séance. Alors nous noue séparons pour 20 minutes, rendez-vous à 11 H 30 dans cette salle. A 11 H 30 exactement nous commençons

RECESS

Président A. MAYER

Messieurs, nous reprenons la séance et je dois vous dire que nous avons reçu de l' - assitance une telle quantité de questions qu'il nous sera impossible de donner la parole aux personnes de l'assistance afin d'y répondre. Nous nous efforcerons de le faire nous-mêmes. S'il y avait des points particuliers que l'un de vous desirerait nous soumettre, nous lui demandons de nous les mettre par écrit pour que nous puissions continuer la discussion.

De même les réponses des membres de l'assemblée peuvent être faites par écrit pour que nous puissions les publier. Je m'excuse de ne pouvoir leur donner la parole. Nous devons travailler avec les Membres du Bureau, et j'espère que vous serez satisfaits de leur travail.

General Reporter S. D. WILSON

To what extent is processing of materials for earth and rockfill dams necessary or desirable?

Panelist A. CASAGRANDE

We have seen in recent years a rapid increase in the use of processed materials in construction of earth and rockfill dams. For example, the Portage Mountain Dam which is the highest earth dam in Canada, was built entirely of processed materials; also the Tarbela Dam in West Pakistan, which is now under construction and which will be the largest earth dam in the world so far as volume is concerned, will be built entirely of processed materials. It is remarkable that for the Tarbela Dam the cost per cubic yard for these processed materials in all zones of the dam is only slightly more than one U. S. dollar. Designers are often too afraid about the high cost of processing. It is true that processing can be very expensive if one finds it necessary to introduce it during construction by a change order. This has happened in connection with several rockfill dams, when the character of the quarried rock did not come out as had been anticipated; and then it became necessary to put all rock over a grizzly and separate the rock fines smaller than a size which is usually fixed between 3 in. and 9 in. Naturally, a negotiated price of such a grizzly operation based on a change order will be excessively high. However, when the grizzly operation is included in the original contract specifications, then the unit cost of putting quarried rock over a grizzly is generally remarkably low. Furthermore, when one separates the rock fines by means of a grizzly, not only does the coarse rock fraction provide cleaner and more satisfactory rock shells, but the rock fines usually provide excellent and wide transition zones between the core and the shells which will replace most or all of the expensive processed filter zones. Finally, the separation of the fines, which otherwise would lie loosely inside the voids of the larger rocks, results in a substantial net gain in total volume in the dam, and this fact alone is usually more than sufficient to pay for the cost of the grizzly operation. For these reasons I am quite sure that we are going to see rapid further developments in the use of processed materials in earth and rockfill dams. In the past we have used materials as they came from the borrow areas, and which were often quite erratic deposits. The designer would write a specification, but he could never be sure what he would end up with in the dam; and in fact quite often the dam as actually constructed was not what the designer wanted. I believe that we are now entering an era in which we will build earth and rockfill dams as we design them, that is we will truly manufacture the materials in accordance with our specifications. Thank you.

Président A. MAYER

Monsieur Casagrande a répondu en premier à cette question. Je crois que Monsieur Pinkerton a également quelque chose à dire sur ce point.

Panelist I. L. PINKERTON

I would generally agree with Dr. Casagrande; nevertheless, I would like to add one or two remarks. First of all, I would like to differentiate between rockfill and gravel fill.

I believe there is a greater case for processing gravel fill than there is for processing rockfill. A rockfill from a quarry, if we endeavour to process it, is often very variable depending upon the place where you are actually excavating in the quarry. In my experience it is better to select the load and to direct this material into certain parts of the dam. If we try to select particularly the fine fraction for any 'cobble transition filter' from the quarry, generally this does not work out too well. I believe that the aim to process is in connection with filter materials where filters have become necessary and by processing we can obtain a filter which we know is going to be satisfactory and therefore we can use the minimum amount of material. The cost of this processing, as Dr. Casagrande says, is quite small.

Président A. MAYER

Je crois que nous pouvons passer à la deuxième question. Monsieur Wilson va la poser.

General Reporter S. D. WILSON

The next question is directed primarily to Mr. Sembenelli's discussion on metal membranes, and I have here a group of several closely related questions which have been presented to us at the same time.

Regarding the upper reservoir of pumped storage plants, where even small leakage is so important, would metal membranes be suitable for lining, such reservoir and floor, and if so, would this be economical? Have you had any experience with the use of PVC or nylon sheeting instead of metal? What about the expansion of metal lining during temperature changes, and what are the details of the joints.

Président A. MAYER

Je crois que Monsieur Sembenelli sera obligé de nous faire un cours sur les revêtements métalliques, mais je crois qu'il peut très bien le faire, et que nous l'écouterons avec grand intérêt.

Panelist P. SEMBENELLI

I will try to squeeze these four questions as in little time as possible. As far as pumped storage reservoirs are concerned, I believe that any scheme with upstream impervious membrane is close to the optimum solution, in fact in such reservoirs we have rapid variations of water level which increase the dangers connected with drawdown conditions; dams with upstream membranes are not affected by drawdown. We also have easy inspection because the reservoir is frequently emptied or near so. There is no doubt that a steel membrane could provide a good solution. As far as costs are concerned I could indicate a cost of 25 dollars per square meter which I think is comparable with the cost of other linings, especially asphalt concrete, recently built in Europe or in North Africa. One of the organizations that used largely this type of structures is the Ruhrtalsperreverein who recently built two dams with this type of membrane, one of such dam was completed three years ago in Italy. I do not believe that the overall facing cost has been less than 20 to 25 dollars per square meter. Going on to PVC membranes, I know only of one PVC application in a small dam in Costa Rica. I think an 8 mm or 10 mm thick PVC sheet was used for waterproofing the dam and part of the reservoir bottom PVC sheets were about 2 m wide and glued together with special glue. As far as I know there are no records on this dam, but it has performed satisfactorily.

About deformations of the metal sheets, I can say that in general, metal membranes undergo horizontal extension close to the crest of the dam, because of downstream deflection of the crest itself. They also undergo horizontal extension close to the toe of the dam because of spreading of the rockfill and of course, in between there is an area where extension is very limited. Dealing with a low dam with a long crest, this extensions are taken usually by vertical accordion joints. The membrane is fixed to the bottom and free at the crest so that vertical extensions can take place unrestricted. In case of narrow valleys, or higher dams, a series of joints which criss cross each other can be used, joint is continious only every two cross points so if we take the whole point grid, we have a double possibility of horizontal and vertical extensions. This has proven a good scheme for providing almost complete flexibility of the membrane. Thank you.

Président A. MAYER

Je remercie Monsieur Sembenelli de ces très intéressants renseignements, et j'espère que les personnes qui avaient posé cette question ont reçu satisfaction. Je passe la parole à Monsieur Marsal qui a une question à poser.

Panelist R. J. MARSAL

What is the degree of hazard which exists from cracking, in view of the increasingly high dams now being built?

Panelist A. CASAGRANDE

The reason why I volunteer to answer this question is because I have been given credit for having invented cracks in dams long ago, i.e. for imagining something that does not happen in reality. But as the height of dams increased, the frequency of cracking increased also, and cracking of dams is no longer a figment of my imagination. I believe that for very high dams with steep abutment slopes, it is not possible to prevent entirely the formation of substantial tension zones and transverse cracks in the top of the dams in the vicinity of the abutments, no matter what materials we use in the dam. Therefore, we must defend ourselves against the effects of cracks. I vaguely recall that in a discussion during our 1957 Conference in London, I said that the designer may proudly proclaim, or believe, that he has designed a dam such that it will not crack; but that he should swallow his pride and design his dam in such a manner that it will not fail even if cracks do develop, not because of his fault but because others sabotaged his dam, e.g. nature. To ensure such safety, the designer will provide several lines of defense against cracking. The most important defense lies in the selection of materials which are self-sealing, particularly in a wide zone downstream of the core, and if at all possible also in the core. In addition, a designer may employ such supplementary defensive measures as upstream arching of the dam, widening of the core and crest in the vicinity of the abutments, i.e. where tension zones are anticipated, etc. Just very recently Mr. John Lowe, of Tippetts-Abbett-McCarthy-Stratton, the designers of the Tarbela Dam, proposed the use of some kind of steel tension reinforcement in the top of the dam within a local zone where we are anticipating transverse tension cracks. The fact that such reinforcement will rust out in the course of years is not important because its principal use would be during the first filling of the reservoir. Experience has shown that it is the first and rapid filling of the reservoir which causes the development of most cracks. If we could control the filling of the reservoir, and particularly fill the top 50 ft very slowly, we would be able to prevent most of the cracks. Almost all materials in earth and rockfill dams have creep properties that permit adjustment to tensile stresses without cracking, provided the tensile stresses are applied very slowly. Unfortunately, in most dams, particularly the high earth dams, we are not able to control the filling of the top portion of the reservoir; and this is particularly true of dams that serve also for flood control. Thank you.

Président A. MAYER

Je vous avais indiqué, au début de cette séance, qu'il y aurait discussion; en voici une:

1o Le Professeur Casagrande nous a parlé de barrages de 200 m de haut. Je pourrais lui mentionner un barrage qui n'a que 30 m. et où nous avons eu une fissure longitudinale tout de même. Il s'agit d'un barrage homogène construit en sable légèrement argileux très compact.

Nous ne savons pas encore très bien à quoi est due cette fissure. Nous nous demandons si précisément elle n'est pas due a un remplissage très lent de ce barrage et à une progression très lente de la zone humide qui aurait produit des tassements à l'amont tandies que l'aval restait sec et ne tassait pas. Ces tassements ne doivent pas avoir leur origine dans le barrage étant donné qu'il a été construit à 95% de la compacité Proctor modifiée, c'est à dire, à une très haute compacité. Nous pensons plutôt qu'il s'agit de tassements du sol sous-jacent.

2o Le Professeur Casagrande nous a parlé d'un barrage pour lequel on envisageait de donner des résistances au cisaillement aux matériaux du noyau. Je ne sais pas si le Professeur Casagrande a eu l'occasion de voir le film que Monsieur Vidal a présenté sur la terre armée, mais il me semble que c'est exactement le procédé qui conviendrait et je crois qu'il y aurait grand intérêt pour Monsieur Vidal s'il est dans la salle, à prendre contact avec le Professeur Casagrande.

Est-ce que quelqu'un d'autre veut parler sur cette question...M.Wilson?

General Reporter S. D. WILSON

Dr. Casagrande should accompany me on Monday to a dam of 150 m. in height with no cracks. The reason I think that this particular dam, which is Malpaso Dam, shows no cracks despite the development of large horizontal tensile strains in the crest, was due not only to the fact that the Dam was built slowly, but more importantly, the reservoir was filled very slowly over a matter of 2 to 3 years. I certainly agree that the hazard of cracking exists and has to be designed for, but certainly there are measures which you can take to minimize the danger of cracking.

Panelist A. CASAGRANDE

I would like to reply to Professor Mayer's comments and questions. I will start with his last comment, concerning reinforced earth. This is a very interesting development. An engineer of that French firm, which has introduced the reinforced earth, visited me a few months ago and showed that film also to my students. It was an excellent film and all who saw it were enthusiastic about it. When John Lowe proposed to me the use of reinforcement in the Tarbela Dam, I suggested that he also review this French development. Concerning the longitudinal cracks in an embankment which Monsieur Mayer mentioned, I would like to say that my previous remarks referred only to transverse cracks. In general I am not afraid of longitudinal cracks, except when they are an indication of a shear failure or slide. There are many different causes for longitudinal cracks. For example, Prof. Marsal could tell you about a dam which was built many years ago in Mexico on a loose foundation stratum that had properties similar to loess soil. As soon as the foundation became saturated during the first filling of the reservoir, severe longitudinal cracks developed in the upstream slope. I do not know whether this information is confidential, and perhaps I should apologize for having mentioned it.

Panelist R. J. MARSAL

May I add another comment on cracks? You have mentioned transversal and longitudinal cracks. There is another type of cracks of particular concern, namely: horizontal cracks due to differences in compressibility of the core, transition and rockfill materials. These horizontal cracks may be dangerous. I hope that through analyses like that of the finite element, using linear or non linear elasticity, those places of the dam that require special treatment could be detected or/and the adoption of second lines of defense be adopted.

Président A. MAYER

Merci, Monsieur le Professeur Marsal.
Est-ce que quelqu'un veut encore ajouter

quelque chose...M.Sembenelli?

Panelist P. SEMBENELLI

I would like to ask a question to Dr. Nichiporovich. The question is dealing with what he was saying before:

Has the method of calculation of core deformations described been applied to dams where actual core displacements are being measured?

Panelist A. A. NITCHIPOROVITCH

Longitudinal deformations in dams like Serre-Ponçon, Peruca, El Infiernillo and others were caused by compression of rockfills that extended deep down to the foundation. The method of calculation applied to several examples, has produced results that are in good agreement with corresponding measurements, particularly in the case of El Infiernillo Dam.

Président A. MAYER

Je pense que Monsieur Casagrande a quelque chose à dire à ce sujet.

Panelist A. CASAGRANDE

I was very much interested in Prof. Nichiporovich's presentation because we have also used successfully the finite element method, particularly for analyzing cracking in earth dams. By analyzing several earth dams which have developed transverse cracks, we were able to demonstrate that the actual location of these cracks agreed rather well with the locations where we obtained the maximum tensile stresses by means of the finite element method. These investigations have been carried out by Sergio Covarrubias for his doctoral research, a young Mexican engineer who is sitting here in the audience. His thesis is now being printed and will soon be available. I am convinced that such finite element analyses will be useful to designers as a guide to determine qualitatively, not quantitatively, where critical zones are located and whether certain measures, e.q. shaping of abutment slopes, would be sufficiently effective to make the expenditure for such measures worthwhile.

Président A. MAYER

Monsieur Pinkerton a une question à poser.

Panelist I. L. PINKERTON

May I ask a question which might be detailed:

When applying this method do you consider the core alone or the entire body of the dam?

Panelist R. J. MARSAL

The entire body of the dam.

Panelist I. L. PINKERTON

The question I have is directed to Dr. Marsal's presentation. At the Malpaso Dike No. 2 a great deal of instrumentation was installed. The results showed negligible movement. The question is this:

Was this much instrumentation necessary and secondly how much instrumentation should be installed in small dams?

Panelist R. J. MARSAL

I believe that it is easy to comment on the subject when the results of observations made at Dike No. 2 of Malpaso Dam are already known. But I would like to ask Mr. Pinkerton: What would be his attitude if he had to design this particular embankment taking into account the uncertainties put forward by the existence of a main fault at the foundation?

Panelist I. L. PINKERTON

This really puts the question back to me. The designer of a dam is certainly faced with this situation. Is he going to instrument a dam to make sure that when the dam is complete its behaviour is exactly as he designed it? Now, I believe we should put the dams in two categories: 1) When a dam has virtually no special problems, a dam where materials being used have also been used on many previous works, that is with respect to the type of materials, and where foundations do not present any unusual features - under these conditions I believe instrumentation should be kept to an absolute minimum, such as surface measurements or some small amount of instrumentation. However, to do that, one must be very sure that there is sufficient investigation to indicate that there are no special problems. This is most important. If investigation should show that there are special problems, I believe there is every justification for instrumenting that particular section of the dam to determine the actual behaviour and to compare it with the designer's assumptions.

Panelist R. J. MARSAL

From what you have already said, Mr. Pinkerton, I would conclude that you agree that the amount of instrumentation installed at Dike No. 2 of Malpaso was needed. Before we started the design of this structure, we did not know anything about the behavior of the foundation. The fault that crosses at the dike-site is about 100 km. long, has a relative displacement of 200 m, and no information was available on its activity. I my add that, if we had to design again this dike, we would put that many instruments, perhaps more, and with a better distribution.

Panelist P. SEMBENELLI

I am going back to what Mr. Pinkerton said, I think everybody agrees that the cost of instrumentation in a dam, especially if it is a large dam, is a minor item. Sometimes this instrumentation is well designed and studied, it can even be reduced below the amount which it is usually accepted with these premises. I would like to say that we have basically two problems, one is to cope with special feature dams of difficult and delicate design: there, no doubt, we need instrumentation, but I also believe everybody would be glad to know how a dam without special problems, but rather close to an ideally simple case, would behave, just to see if our computation or what we can forsee with mathematical analysis, or other means, is true. I would say that we need instrumentation in any case. If the dam is simple, it will help to support our design procedures which are largely idealized. If the dam is complicated, it will help us to know what is going on.

Panelist R. J. MARSAL

I disagree with Mr. Sembenelli. I do not believe that every dam should be instrumented. The amount of work that has to be done to collect the data, to process them, and make interpretations is such that it would result a tremendous task of doubtful value in most cases. I think we have to choose carefully, for academic purposes, a dam or two, instrument them thoroughly and have the courage to go ahead for years to observe their behavior with particular care. On the other hand, when the engineer faces a special foundation or embankment problem, he has to resort to appropriate instrumentation, in order to forsee events that may call for additional remedial measures before a malfunction develops.

Président A. MAYER

Je crois que je pourrais faire une remarque ici, c'est qu'il y a dans les petits barrages dont il est question ici, beaucoup de cas où si l'on voulait mettre des instruments à l'intérieur du corps du barrage, on ralentirait considérablement la construction, et où le coût de pose des instruments serait tout à fait négligeable par rapport au ralentissement du travail que cela produirait. Vous avez tous vu tout à l'heure la photo de ce garde que l'on était obligé de mettre sur le chantier à l'intérieur de barrières, entourant un instrument que l'on avait mis dans un barrage. La présence de ce garde ne facilitera certainement pas le compactage, s'il évite la mort; au contraire, il le rendra plus difficile exactement au point où on voudrait qu'il soit bien fait. Je crois que dans tous les cas relativement simples, s'il est indispensable de placer des repères topographiques - après construction de façon à savoir ce qui se passe exactement, s'il est indispensable également, après construction, de mettre des piézomètres pour surveiller le niveau de la surface de saturation à l'intérieur du barrage. Il ne faut placer des instruments à l'intérieur et prendre le risque de ralentir la construction que lorsqu'on se trouve dans des cas spéciaux, dont le Professeur Marsal avait parlé tout à l'heure. Bien sûr, lorsqu'il y a une fissure de 200 m de profondeur, il faut savoir ce qui se passe, même si on ralentit le travail. Mais dans beaucoup de cas la construction ne pose pas de problèmes. Alors, laissez travailler l'entrepreneur, et ne constatez les résultats qu'après coup. Monsieur Wilson prend la parole.

General Reporter S. D. WILSON

I would like to add one more comment. In one of my charts in my paper, you will remember that Mexico was shown to be the leader in the percentage of rockfill dams being constructed. I would like to add to that point. In my opinion, Mexican engineers are also the leaders in the instrumentation of dams, in the development of instrumentation, and in the proper interpretation of the results.

I have a question from the audience that says, "My experience indicates that some dam designers favour types of dams with which they are familiar, regardless of the geological, topographic or other conditions prevailing at the site."

"How can we make the profession aware of the need to use a more flexible approach?"

I certainly agree with this comment. Would somebody else like to add to this?

Panelist A. CASAGRANDE

This is a problem in human engineering not in soil engineering.

Président A. MAYER

J'ai une question maintenant qui m'a été posée en français. Je vais donc la lire en français, "Le placement des matériaux et leur compactage dans les barrage en terre causent une stratification.

Quels sont les effets de celle-ci sur les infiltrations à l'intérieur du barrage et nécessitent-elles des dispositions spéciales telles que filtres, ou drains verticaux?"

Je crois que c'est une question sur laquelle les Membres du Bureau seront disposés à répondre.

General Reporter S. D. WILSON

Let me just re-read that question. The translation from the French was not too precise. The placement and compaction of earth and rockfill materials in layers causes stratification with respect to permeability.

What effect does this stratification have on the seepage path and does this require special design steps?

Président A. MAYER

Je voudrais donner à cet égard un avis parce que j'ai eu précisément le cas d'un barrage dans lequel le compactage avait causé une différence considérable de la perméabilité horizontale et verticale. Nous avons constaté que le fait de mettre comme habituellement un filtre à la base de la face aval, n'était pas suffisant pour obliger la surface de saturation à rejoindre le filtre. Il y aurait eu avantage a prévoir soit un filtre vertical soit des drains verticaux, comme j'ai vu avec intérêt qu'il en avait été placé dans certains des ouvrages dont nous avons eu les coupes tout à l'heure au tableau. Monsieur Wilson répondra également à cette question.

General Reporter S. D. WILSON

The design of filters in transition zones for non-cohesive soils is well established. Does the panel agree that requirements for filters for cohesive soils are different? Would Dr. Casagrande comment on this?

Panelist A. CASAGRANDE

This question is also intimately related to the transferal cracks in the dams because the filters are supposed to protect the core and if the core is of non-cohesive material, very silty sand, or sandy silt, then the filter has one type of function if; we are dealing with a real clay in the core then the danger of cracking is often more severe and in any case we would then have the function of the filter to protect against crack and not against percolation through the clay or soil. As a matter of fact, I am not afraid at all of the water that percolates through the clay core, if there are no cracks. So there are differences in functions and perhaps even differences in design if we keep it in mind. Thank you.

Président A. MAYER

Professeur Marsal pose une autre question?

Panelist R. J. MARSAL

Yes, I have another question which was brought up by the audience and it says, "What would be a sound criteria in regard to the ratio of head of water divided by thickness of core?"
This man is asking, as time has passed, we are building rockfill dams with thinner and thinner core. Hence, this man asks what would be a sound criteria for the ratio of the head of water to the thickness of the core at the base.

Would you be willing, Dr. Casagrande, to answer this question?

Panelist A. CASAGRANDE

Well, in my experience, it is not really true that the core of rockfill dams are getting thinner and thinner. There was a perriod some 10 or 15 years ago when some of the rockfill dams with inclined core were built with very thin core, in fact, so thin they scared me, but the trend has been away from this very thin core and the high rockfill dams which I am familiar have generally a ratio of widths to the hydraulic head of no less than 30%. The corps of engineers for example in the United States uses an absolute minimum requirement-20%, but I do not know of a single dam in recent years in which they have gone that low, 25% is perhaps the minimum. Thank you.

Panelist P. SEMBENELLI

I believe that the design of an earth dam is largely dependent and almost the result of an equation where available construction materiale enter as a paramount term. According to the characteristics and volumes of available core materials, one should vary the thickness of the core.

Panelist I. L. PINKERTON

I would just like to add a few remarks on this question. One, of course, could say theoretically that the amount of seepage that passes through the core would vary with the thickness. However, other considerations may prevail. Over the years, we have used in Australia a figure of 33%. In addition, I agree with Mr. Sembenelli: it is a question of variability of material and if sufficient materials are available I see no point whatever in putting in a very thin core.

Président A. MAYER

Messieurs, voici je crois à peu près une heure que nous répondons aux questions qui nous ont été posées. Il en reste d'ailleurs un certain nombre mais je crois que vous nous avez assez entendus et que dans ces conditions nous allons clore la séance.

M. COLL et J. M. PEIRONCELY (Espagne)

Un barrage en terre est en construction sur la rivière Manzanares, juste en amont de la ville de Madrid. Le but de ce barrage est de régler les eaux de cette rivière dans sa traversée de la capitale d'Espagne. On construit sous la direction des ingenieurs de la Confederación Hidrográfica del Tajo, Ernesto de Jaureguizar et Manuel Sanz Martín.

Le barrage a une hauteur de 25 mètres et une longueur de crête de 700 mètres.

Le barrage est composé d'un matériel formé par des sables argileux ayant un noyau imperméable en amont, une couche de protection dans le parement et un filtre de graviers.

Le barrage de El Pardo est situé sur des terrains sableux du Miocène Continental provenant de l'érosion des massifs granitiques. Ces terrains sont composés de sables plus ou moins argileuses.

Ces dépôts de sables sont postérieurs aux movuvements tectoniques. Ils ne sont pas plissés mais présentent une légère inclinaison au SSE qui atteint, parfois, 3%.

Le Miocène est, en partie, recouvert de dépôts quaternaires.

Les sondages réalisés, même ceux d'une grande profondeur, n'ont pu prendre contact avec les terrains inférieurs. Dans le Miocène, nous avons distingue cinq sortes de terrains:

1°.- Sables compacts argileux

2°.- Sables argileux supérieurs avec une épaisseur de 7 a 10 mètres.

3°.- Sables intermédiaires dont le contact est très clair avec le 2°. Epaisseur de 13 à 14 mètres.

4°.- Argiles inférieurs de 10 à 12 mètres d'épaisseur.

5°.- Sables inférieurs.

Tout cela apparait dans la figure 2 (coupe géologique)

Les essais de perméabilité, du type Lefranc, donnent les coefficients suivants: K= 10 10^{-8}. Les essais par gravité donnent des valeurs un peu inférieures. Les mesures de perméabilité donnent des valeurs de l'ordre de 10^{-4}. Mais ces perméabilités ont été faites dans le sens vertical.

Le Quaternaire est très perméable mais la perméabilité diminue du tronçon I au II. Le tronçon III présente des voies artésiennes.

L'orgine des voies artésiennes se trouve dans la pente des tronçons qui les fait affleurer (Figure 3-croquis géologique). L'eau est canalisée dans les niveaux les plus sableux; c'est-a-dire qu'il se produit des filtrations importantes dans les tronçons III et V.

En raison de l'affleurement de ces tronçons dans le barrage, le probleme s'aggravera lorsqu'on le remplira.

La pression actuelle dans le tronçon III est de 7,4 m. de colonne d'eau. Lorsque le barrage sera plein, elle sera de 31,4 m. Le poids du barrage résistera a cette poussée mais, en aval, nous ne pouvons compter que sur un poids équivalant a 13,65 m. et la rupture du terrain pourrait avoir lieu.

Le tronçon V aura une pression de 56,4 m. et un poids de résistance équivalant à 63 m.

En outre; en augmentant les pression les fines pourraient être entrainées, ce qui provoquerait une augmentation de la perméabilité.

Pour pallier ces difficultés, un écran de béton a été prévu pour prolonger jusqu'en bas le noyau imperméable qui atteindra le tronçon II.

Evidemment; cet écran diminue mais n'évite pas l'ecoulement des eaux. Pour éviter cela, le projet prévoit, en aval, une protection en enrochement de 80 mètres (Figure 1).

En outre, un drain efficace permettant sa révision périodique doit également fonctionner en aval. Avec ce drain; on évite toute surcharge d'eau et l'entraînement des grains fines.

Les drains prevus sont des puits (Figure 4) de Ø = 600 mm. ayant des tubes perforés et une grille du type "Johnson" de Ø 300 mm.; la couche filtrante intermédiaire est faite de graviers et de sables.

Le calcul du nombre de puits nécessaires - c'est-à-dire de leur séparation - a été fait dans l'hypothèse d'une perméabilité horizontale dans le troncon III dix fois supérieurs a celle qui a été évaluée par les essais. Nous avons appliqué la méthode des images pour

Les trois puits du lit donneront ainsi un débit de 21 litres par seconde.

S'ils étaient insuffisants; de nouveaux puits pourraient être intercalés.

Des piézomètres seron placés dans les points intermédiaires afin de vérifier les pressions

une descente minimale admissible de 17,8m. La séparation qui en a résulté est de 25 mètres.

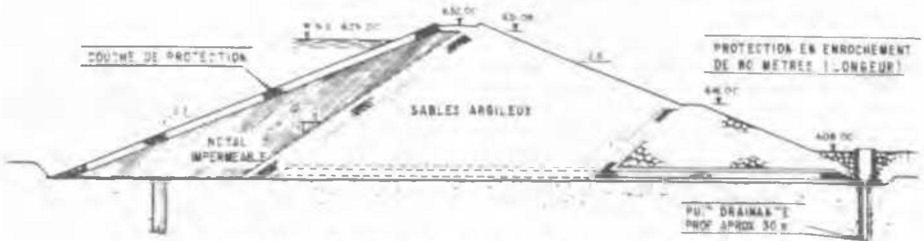


Fig. I Profil — type du barrage

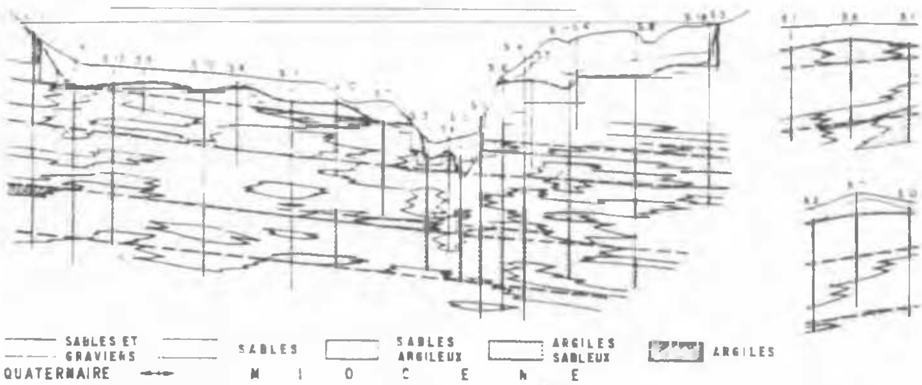


Fig. 2. Coupe géologique

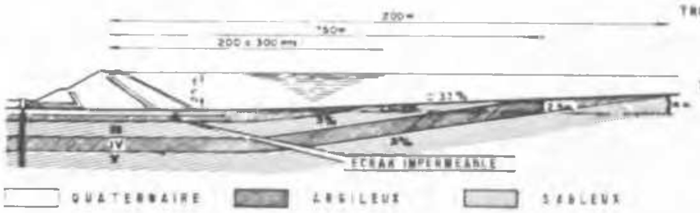


Fig. 3 Croquis géologique

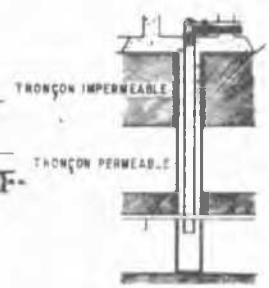


Fig. 4 Puit drainante

R. CORDA et M. F. BOLLO (France)

Pour compléter l'état actuel des connaissances, il semble utile de mentionner l'existence de méthodes de contrôle de l'efficacité du traitement des fondations perméables permettant également de suivre l'évolution des infiltrations dans le temps.

Il est possible dans ce but de faire appel à un réseau d'electrodes fixes utilisé avec succès au barrage de KRUTH-WILDENSTEIN sur la Thur (France).

La coupure étanche dans les alluvions en fond de vallée comporte un écran injecté de 15 mètres de hauteur maxima prolongé au rocher d'une longueur égale et poursuivi sur les flancs.

Immédiatement à l'amont et à l'aval du rideau ont été placées des électrodes impolarisables distantes les unes des autres de 7 à 20 mètres, reliées à un tableau de mesures placé dans la cabine de commande et de contrôle du barrage. Une électrode était située à l'infini.

La résistivité du terrain a été mesurée retenue vide et au cours du remplissage de barrage, entre diverses combinaisons d'électrodes. Il a ainsi été possible de déceler certaines anomalies dans les variations de résistivité, au fur et à mesure de l'élévation du plan d'eau.

La continuité du voile d'étanchéité a été contrôlée d'autre part par un ensemble de mesures sismiques qui ont donné des informations satisfaisantes, au rocher notamment.

Ces déterminations, recoupées avec le résultat des mesures d'autre nature habituellement pratiquées, ont permis de tracer la carte des anomalies du traitement des fondations et d'intervenir avec efficacité aux emplacements voulus pour parachever le rideau d'étanchéité.

D'autres applications ultérieures ont fourni de bons résultats dans des conditions géologiques assez variables telles que les suivantes:

Ecran d'étanchéité de 4.5 kilomètres de longueur de la raffinerie B. P. d'IGOLSTADT.

(Plaine alluviale du Danube - R. F. A.)

Barrage de CASTRELO (Rivière la Sil, Galice-ESPAGNE), sur des arènes granitiques et du granit altéré.

Un tel dispositif de mesures électriques, complétant les mesures géophysiques, sismiques en particulier, paraît susceptible de rendre les plus grands services pour le contrôle de l'exécution du traitement des fondations perméables à diverses profondeurs.

C. M. GUILFORD (Hong Kong)

In section 5.4 of their paper, the Reporters have stated that conventional methods of constructing a dam are sometimes impracticable. This is clearly so in the case of a dam built in the sea. In this connection, I wish to illustrate three interesting aspects of the construction of the 6800ft long Plover Cove marine dam in Hong Kong (see State-of-the-Art Volume, page 200 and Proceedings Vol.2, page 291), namely the final sealing off of Plover Cove from the sea, the nature of original seabed and completed dam.

Prior to closure, two 1800ft lengths of dam adjacent to the abutments were raised through the tidal range and the central 3000ft section of the dam was built to a constant level of 20ft below mean sea level. The various phases of the closure operation are shown in fig 1 and are described below:-

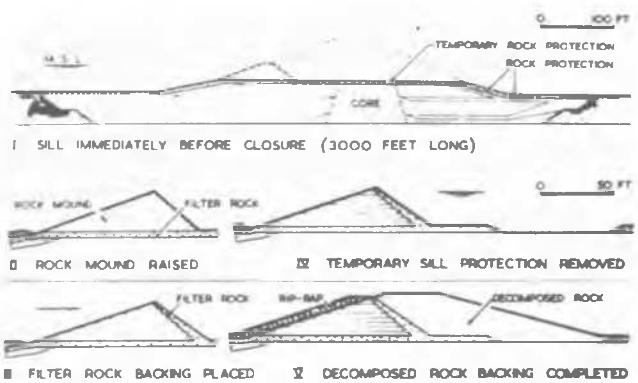


Fig 1 Stages in closure operation

Stage I - The toes, sloping faces and sill of the dam were all protected with rockfill to prevent possible erosion during the final rock closure operation. At this stage, maximum tidal velocities of about 1.2ft/sec were expected which approached the limit at which the principal embanking material, decomposed granite, was stable. No protection to the seabed mud was considered necessary.

Stage II - The rock closure mound was formed in horizontal layers of approximately 3ft thickness extending the full length of the closure gap by means of a large floating grab placing pontoon equipped with twin-22yd^3 buckets - see fig 2. As the mound grew in height, so reducing the area through which seawater could pass (tidal flows up to 85000 cusecs), currents increased. The maximum observed velocity of 9.7ft/sec occurred in a low spot when the crest of the closure mound was around mean sea level and compared closely with the maximum recorded on a tidal model. No significant movement of rockfill or seabed mud took place during this phase.

Stage III - The downstream face of the closure mound was backed with a graded rock filter by means of grabs mounted on marine plant.

Stage IV - The temporary rock protection to the core of the dam was removed with a bucket dredger.

Stage V - A decomposed rock backing to the completed rock mound was placed by land and marine plant - see fig 3; in the photograph

Fig 2 Final rock closure

Fig 3 Backing operation nearing completion

Fig 4 Turbulence caused by trapped air in recently placed fill

will be seen clouds of decomposed rock fines, washed out of the backing, flowing through the completed rock mound.

Fig 4, portraying a close-up of the decomposed rock backing to the rock closure mound, clearly shows turbulence and migration of fines caused by trapped air escaping from the recently placed fill. This process probably accounts, in part at least, for the loss of fines and segregation which had previously been noted in the decomposed granite placed with grabs under water in the core of the dam.

Transverse cracks were noted in the one section of the backing - see fig 5. These were believed to have been caused by differential settlement and lateral movement of the soft underwater fill material. The effect of these cracks was not considered serious as part of the backing was to be reworked subsequently. (Even if cracks remained, in view of the well-graded nature of the decomposed rock and adequate thicknesses of filter layers, no piping would be likely.)

Fig 5 Transverse cracks in temporary backing

As soon as the backing was substantially completed, some 25000m.g. of trapped seawater were removed by means of a 250m.g.d. floating pumping station. Fig 6 shows a general view from inside the empty reservoir. In the foreground can be seen the original test mound, successful completion of which proved that it was possible to form under water a stable and impervious fill using decomposed granite as the principal embanking material; in the middle distance, stands one of the three marine gauge houses.

Fig 6 Empty reservoir

Fig 7 Desiccated mud surface

Fig 8 Completed Plover Cove marine dam

As the mud (normally consolidated clay) dried out, cracks formed on the seabed - see fig 7. These were generally pentagonal in shape, 18in. across and 9in. deep and effectively increased the mud surface area. During the first filling, due to diffusion, the cracks gave rise to a more rapid increase in the salt content of the stored fresh water than originally anticipated. In the first two years of operation, the salinity (dissolved salts) has dropped from a maximum of 2250p.p.m. to 370p.p.m. (September 1969).

So successful has the completed dam (see fig 8) performed that plans are now in hand to raise the water level in the reservoir by some 16 feet (i.e. 10000m.g.).