SPECIALTY SESSION 3

DESIGN OF EARTH AND ROCKFILL DAMS

Chairman: V.S.Eristov (USSR), Vice-Chairman: E.G.Gariev (USSR)
Participants: Stanley D.Wilson (USA), A.O.Uriel (Spain), John Lowe (USA), Comsa R.S. (Romania), T.F.Lipovetskaya (USSR), P.Habib (France), Vlad D.Perlea (Romania), Hosking A.D.(Australia), Serge J.C.Hsu (Brasil), Akimov A.A. (USSR), Burenkova V.V. (USSR), Mojevitinov A.L. (USSR), Avakian L.A. (USSR)

Chairman Prof. Eristov V.S.

Ladies and Gentlemen! Dear colleagues and comrades!

First of all let me greet you at our special section No.3. Before starting our session, I request all those present to stand in memory of our colleague, who has passed away prematurely, I mean the immediate past-president of our specialty session, one of the outstanding specialists in the field of design and construction of earth and rockfill dams, a member of the Soviet association in soil mechanics and foundation engineering, professor A.A.Nichiporovich. It is a tragic loss for the Soviet Science.

(Members of the Conference stood for a few moments).

Thank you.

The activities of our section will be devoted to calculations and investigations of the earthfill and rockfill dams. Unfortunately we are lack of time, though we have two and a half hours and we must use it as much fruitful as possible.

You are aware of the fact that construction of the dams of this type develops intensively all over the world, the possibilities for their erection in different geological conditions are great, and therefore they become the most economical in comparison with all types of concrete dams.

This explains the fact that 90 percent of all the dams constructed in the USA for the last years are either earthfill or rockfill dams with clayey cores and blankets, in Canada this percent equals to 67.

In the USSR more than one hundred and fifty dams of this type, more than 25m high, have been constructed or under construction now including such dams as recently constructed or being constructed the Viljui dams 75m high, the Sion- 86 m high, the Medeo, 115m high, the Kolymsk dam, 130m high, the Charvak dam, 168 m high and the highest in the world the Nurek dam more than 300m high. The Soviet Union designed and participated in the construction of the High Aswan Dam on the Nile, 111m high. Recently, in the USSR the Hydroproject Institute finished designing the Rogun dam on the Vakhsh River, the height of this dam will be 325 m.

The height of the earth and rockfill dams constructed to-day in different countries of the world are also rather great, e.g. the Messaure dam in Sweden, 100m high, the Miboro in Japan, 125m high, the Serr-Ponson in France, 130m high, the Madmountain dam in USA, 130 m high, the Infiernillo in Mexico, 148m high, the Gepatch in Austria 153m high, the Gesheneralp in Switzeland, 156 m high and one of the highest dams in the world, the Oroville dam, in the USA 224 m high and Mika-Creek in Canada, 235 m high.

Therefore, at the construction of such type of dams and of a comparatively low heights no difficulties of a theoretical character have been arosed before but to-day the heights and heads are so big and the volumes amount to dozens of millions cu.m. very serious and important problems arise, many of them have not been solved yet. The right solution of them is an extremely important matter for the modern dam construction and here we stumble upon the problem of the soil mechanics. To this we refer the following:

1. Calculations of dams stability and its parts with regard to the pore water pressure in the clayey cores and blankets of the rockfill dams and in the body of the homogeneous dams.
2. Analysis of consolidation of the clayes cores and blankets.
3. Determination of strain-stress condition of the dams and their subsidence.

4. Revealing the relationships of the interaction of the cores and blankets of the dams with the shells.
5. Measures to prevent from cracking in the cores and blankets.
6. Filtration calculations taking into consideration crack formation in the cores and the impermeability of them.
7. Methods of consolidating the cores, screens and shells in the process of erecting them.
8. Seismic effect influence on the stress and strain condition of the dams and their stability.
9. Methods of determining the strength and deformation characteristics of materials for cores, blankets and riprap.

In connection with these problems it is necessary to carry out a wide range of scientific studies both theoretical and experimental, including the field studies in the process of dam construction and after it and correlation of the results of the analysis and experiments.

In the Soviet Union due to the great prospects of the hydraulic structures construction including in the seismic areas much attention is paid to these questions.

A number of scientific and research centres are busy to solve these problems, among them: the Hydroproject Institute named after academician Zhuck, the Hydrotechnical Institute named after academician Vedeneyev, in Leningrad, the Institute "Vodgeo" in Moscow, the Moscow Engineering Institute named after V.V.Kuibishev, Georgian Scientific and research Gruzniiges in Tbilisi and others.

Very important is the exchange of experience on an international scale and scientific cooperation of all countries. And the activities of our section of the VIII-th Congress are devoted to this task.

A whole number of rather interesting and important reports have been sent to the secretariate of our section on the above mentioned subject from the scientists of different countries, altogether there are twenty three reports, including one report from Australia, one- from Great Britain, four-from India, one-from Mexico, three-from Poland, one-from Purtugal, one-from Roumania, two-from France, one-from Chockoslovakia, one-from Sweden, one-from Japan and seven reports from the USSR. Four contributions of Soviet specialists have been already published in the Collection in russian.. All contributions will be published in the Conference Proceedings in english.

Some of the authors of these reports will report here the results of their investigations and their points of view. In a free discussion we shall exchange the experience of the job we do and shall exchange our opinions.

The acquaintance with the received papers demonstrates that they all are a valuable contribution in our science and improve our knowledge.

They show specifically that in order to perform calculations on the stress-and-strain condition of the dams the method of finite elements with the use of computers has found a wide use in many countries-such are the reports of Eng.Toshikatsu Kawamota, Tetsura Nivam, Tamonabu Miyabuta (Japan), of Dr.Katti and Sandram (India), Eng. Dolezalova (Chechoslovakia), Eng. Constantinesku (Romania). Eng.Habib, Luong, Nguyen (France), eng. Penman (Great Britain) M.S.Rasskasov, Vittenberg and others (USSR), in which the nonlinear-elastic-plasticity problem is being solved.

The major part of these reports is devoted to correlation of the results of model and fields tests with the design ones.

It is evident that the method of finite elements permits to consider all the necessary initial data and good correlation with the experimental data and a sufficient accuracy of the tasks solution is obtained.

A number of reports deals will the influence of pore water pressure on dam stability-reports of engineers Mr.Dupa, Fari and Londe (France) and D.S.I.Fyodorova, Eng.Dergachova and M.S.Alipova (USSR). The engineer Allystein from Mexico proposed a new method of calculation of dam subsidence, the engineer Kolosov M. (USSR) proposed a method of calculation of the upstream shell doformation and compared it with the results of field observations.

M.S.Kryzhanovsky A. and Eng.Chevikin A., Pikus Y.E.and Pashenko M.(USSR) analyse the influence of deformation behaviour perculiarities of the soils upon the stress-and-strain condition of dams using the means of the non-linear rock mechanics, eng.Ryshenko A. (USSR) reports on the influence of stress condition of gravely soils upon the angle of internal friction. The soviet specialists, M.S.Vutsel V. and Eng.Sherbina V. investigated cracking in the core of a rockfill dam by a method of centrifugal modelling.

A number of reports contains interesting data of the field survey on the behaviour of the dams and banks-eng.Marane de Neves and Guedes de Melo (Purtugal), Dr.Fiedler and eng.Wencewisc (Poland).

The reports of eng.Ramamurti, Kanikshar, Prokash and Chavla (India) and eng.Kogan P. and Zalezhnev Yu. (USSR) contains the data on the behaviour of grained and coarse debris rocks at low and high squeesing pressures and eng. Tanikichalam and Saktivadavely (India) present the method of calculation of the transition zones.

At last in the reports of the Indian engineers Shamshera Prakash, Handkumaran and others the problem of seismicity influence upon the stability of the slopes of earthfill dams is looked upon and a method of model dynamic studies on vibroplatforms is being proposed.

The problems of dynamic effects on the dams have been already touched upon in the above mentioned report of M.S.Rasskasov L. Vittenberg and others (USSR). As it is seen from my breaf review all the reports answer the subject of our section and are of a certain interest.

I am sure that all the participants in the discussion will also introduce much new and interesting and that the activities of our section will be quite fruitful and useful for all the participants and specialists, dealing with designing, investigation and construction of earth and rockfill dams.

In conclusion I implore my dear colleagues to be stick to the time-limit during their speeches. The time-limit has been fixed by the organizing committee as the following: 7 minutes for the first fixed reports and 5 minutes for the speeches in the free discussion. In compliance with the instructions of the organizing committee I am obliged to follow the way the time-limit is kept to.

Thank You.

Now I pass the word to Vice-Chairman Mr. Rasskazov.

M.S.Rasskazov L.N. Vice-Chairman

Dear Chairman! Ladies and Gentlemen, Comrades!

The activities of our section are devoted to a rather narrow problem to be considered at the Congress but very wide at the detailed consideration of it. Essentially this is the question that is closely connected with all the fundamental problems of soil mechanics.

The fact that this problem is being considered at a special section of the Congress speaks about its urgency and prospects.

To-day we must state that the numerical methods of calculations of stress displacements and stability of the dams are widely applied at dam designing from the building materials and various approaches for solving the tasks are being used: method of finite differences, and of variational-differences; and a method of finite elements, which is more widely spread compared to other methods here we must express our gratitude to those who have worked it out and introduced, i.e. to Mr. Argiris, Zinkevich and Cheng, Clough, Shorpa, Rosin and to others.

At present there is a great number of studies on the improvement of the method of finite elements, i.e. on the increase of calculations accuracy, etc. These studies are very valuable but we should not forget that all the above mentioned methods- methods of linear theory of elasticity and their application in its pure aspect for the soil structures is dealt with rather serious allowances.

It is necessary to remark that there are some papers in which these methods are applied for solving non-linear tasks by iteration way introducing variable values of Young's modulus and Poisson's factor regarding dam construction in stages but the solving procedure is rather complicated and therefore it has not been so widely spread.

To-day the solution of non-linear plasticity tasks by other methods is very important and it has nothing to do with the concept of linear theory of elasticity. In this respect a method proposed by Prof.Chernousko and developed by his collegues, the method of local variations that can be applied couples with the method of finite elements. This method is less responsive to a character of stress-and-strain correlations. Its new application puts new tasks before soil mechanics:

1. it is required to establish relation between stress and strain, but it is not necessary to apply the deformation theory of plasticity but to use the associated law of flow recording the function of loading depending on the way of loading in a general case, that makes the application of this method very complicated and very difficult to establish relationships between the increments of stress and strain from the experiments;

2. passing from stresses to stability it is required to chose the condition of strength. Classic strength conditions (Mohr-Kulone, Mizesa-Botkin etc.) applied to soil do not take into account the effect of Lode parameters and modes loading that is extremely important.

Here we may use Mizess-Botkin's condition but in its main aspect. The condition of strength is necessary not only for establishing stability factor but as an indicator showing soil state; before limit state or ultimate state when the theory of plastic for comes into force;

3. complex state of stress of soil in a structure changes with the structure growing up and the soil may be variably loaded or unloaded in the points under review. And if we do not apply the associated law of flow then firstly, the indicator showing the loading condition in a given points is required (straining energy increment may serve as an indicator), secondly, it is necessary to know the correlation between stress and strains at the unloading (that may be linear in the first proximity), thirdly, at the repeated loading we must know when we must come back to an initial branch of loading from the linear correlation and possibly a coefficient of material condition at the beginning of loading of (Mizes-Botkin's theory in its pure aspect may serve as an indicator).

All these problems are to be solved by soil mechanics and we should not forget that we meant conventional-instantaneous deformations but there are also questions of creeping of different soils (including coarse debris) under a complex state of stress.

Prof. Lomize G. and his colleagues develop these problems experimentally. We must be aware of the fact that the task solution in this field has not been presented at the section.

Majority of the authors used the method of finite elements very often in its pure aspect and it might be of an interest especially in the paper Kawamoto etc.
(Japan) who correlated the calculations (non-linear task) and model studies and revealed that at the approximation of soil condition to the ultimate condition in a model the difference between the results of analysis and the data of the model tests becomes inadmittably great; the paper gives an explanation to this phenomenon that the deformations of

soil sample investigated in a three-axial device at the compression are being given up, when values E and a are determined. The explanation is connected with the above stated.

Coming to dam stability investigation due to seismicity it is necessary to remark that the essential progress that may be observed in this field is also thanks to the method of finite elements.

To-day we already may successfully find the proper values and proper vectors, pulsation of stresses in different points of the dam, etc. but coupled with this, many new problems have been found:

1. first of all choice of design accelerogram;
2. transition from stress pulsation to residual displacements and dam stability;
3. determination of design characteristics and first of all of the coefficient of decrement in the soils.

Accelerogram recorded during various earthquakes during the most unfavourable one a structure is taken as a design one.

Now we must learn how to plot a design synthetic accelerogram to meet the geological and seismic conditions of the damsite. In "Vodgeo" the transition from stress pulsation to residual displacements is performed by applying the combination design-experiment design. So, for the dam of pebble material and with the inclined core 130 m high, we might expect the residual displacements equalling to ≈ 20 cm due to the influence of 9 degree earthquake. The idea lies in the fact of stress pulsation that have been found from seismic analysis and new values of the Young's modulus and Poisson's coefficient is applied to a soil sample. Further static calculations make possible to reveal the residual displacements.

In this respect the progress may be obtained in case of soil behaviour study with time, when soil is in its limit conditions as it has been proposed by Lyatkher V.M., Didukh V.I. As a whole it may be stated that as present we afford the sufficiently reliable method of dam analysis for the seismic effect that certainly needs to be improved. Comparison of the analysis results with the model tests data due to seismic effect showed a good correlation.

In conclusion I would like to note that the to-day's methods of analysis have left behind the studies of soil properties and therefore a great sphere of action opened before soil mechanics on a qualitatively new basis. Thank you!

Chairman Eristov V.S.
Thank you Mr.Rasskazov. The next will be Vice-Chairman Mr. Gaziev

Erast G.Gaziev, Vice-Chairman

Mr.Chairman,Distinguished Delegates,
I would like to say a few words about the basic aspects of the design and investigations of earth and rock-fill dams.

Lately wide mechanization of construction works, development and use of highly efficient machinery and mechanisms has created favourable conditions for the construction of large earth and rock-fill dams involving considerable volume of earth moving operations. One can trace a distinct tendency of increase of the structures' height which requires special responsability on its design, study and construction.

Problems that are facing designers embrace practically all fields of the soil mechanics, but one should select the most important problems solving which are of primary practical use for the construction as a whole.

Especially it is important for our today discussion taking into account very limited time at our disposal.

Static and filtration stability of the structure and forecast of settlements are the basic engineering problems arising when design and construction of the earth and rockfill dams are being worked out.

In solving these problems numerical methods of calculation find at present wide application, and in particular the Finite Element Method which provides new unlimited possibilities for engineering calculations. However by using analytical methods for analysis of the earth and, especially, rock-fill dams one should clearly appreciate the behaviour of the material under conditions of incipient failure but to achieve this end one should have possibility to track down dynamics of transition to the incipient failure conditions with due regard for consolidation,rheological properties and methods of dam construction mean while the consideration of all these factors makes calculations complicated thus limiting the possibilities of practically using these calculations.

Therefore alongside with the development of new methods of calculation and improvement of the existing methods, simplified experimental and analytical relationships which will permit securing reliable results at minimum cost should be created.

But in all cases one should clearly see the limits within which a method of calculation can be applied.

Since behaviour of the structure under load is the main criteria in assessment of the structure behaviour, which enables corrections of the theoretical assumption to be checked, particular emphasize should be put on the field in-situ observations of built dams.

Systematization, digesting and interpretation of results of the in-situ observations help to apprehand the basic laws and processes occuring in the dam body.

I hope these issues will be reflected in today's discussion.

Another rather interesting and promising problem is a search for new types of earth and rock-fill dams, as well as for new methods of construction.

The Soviet Union has interesting experience in development and application of pin point blasts to create dams in narrow gorges.

One of these dams, built in Medeo, saved last month the city of Alma-Ata, blocking the powerful and most destructive rocky-water flood.

Development of methods of creating grout curtains in rock fills undertaken in France, presents a great interest as applied to the above said dams.

Particular attention should be given to an effective method of reinforcing the earth fills which makes it possible to built earth structures, that is structures of reinforced earth, of a contracted section with vertical slopes.

Recently such structures have been built in France, Italy, Germany, USA, Canada and other countries. In november 1972 Vallon de Bim Dam made of reinforced earth was completed in France. The dam is 10m hight with water spilling over the dam crest. At present a design is in progress for a dam of 40 m hight.

It would be of interest to hear today about new ideas and trends in this field.

Should we be able to touch on and discuss even a part of the presented problems, our session will be fruitful and interesting for all the participants. Thank you.

Chairman V.S.Eristov.
Thank you Mr. Gaziev. Mr.Wilson will you introduce your contribution

Stanley D.Wilson (USA)

STATEMENT OF PROBLEM

Analytical techniques used to predict the ground response of a dam under earthquake loading conditions require the determination and use of the dynamic properties of the embankment materials and underlying soils.

In situ measurement techniques presently in use induce strains significantly smaller than those produced by earthquakes in the strong motion region, while conventional laboratory tests are performed at strains either higher or lower than those produced by strong motion earthquake shaking. This discussion describes apparatus now under development for an in situ test procedure to determine the shear modulus at depth at strains equal to or approaching those in the strong motion earthquake range.

APPROACH TO PROBLEM

A cross-hole procedure utilizing an impulsive energy source is used. The signal generating system consists of an in-hole anchor and striker assembly which imparts an impulse shearing energy to the soil.

The anchor is raised and lowered into the borehole by a tripod and rope shown in Fig. 1.

The objective is to generate a clear repeatable shear signal in a borehole, and to measure in closely-spaced adjacent holes the propagation velocity of the signal and the maximum particle velocity. From these determinations, it is then theoretically possible to calculate both the shear modulus and the corresponding strain level.

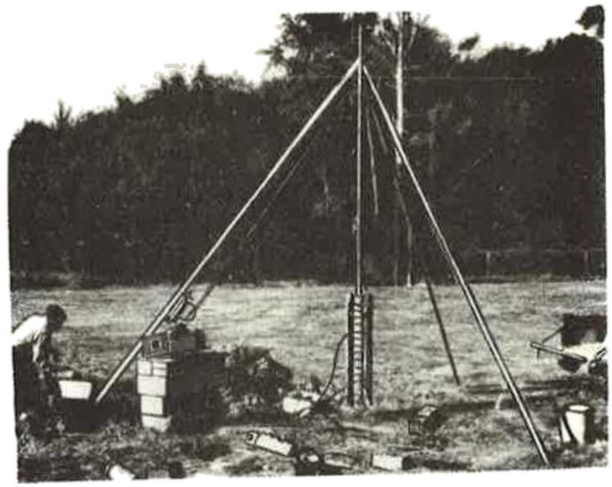


Fig. 1 Soil Anchor and Tripod

ANCHOR AND STRIKER ASSEMBLY

Shear energy is transmitted to the soil by striking with various drop weights an anchor system, which is expanded tightly against the walls of the borehole. The anchor is seen in Fig. 1.

By changing the weight and the height of drop of the striker at a given test depth, the strain levels at which measurements of shear velocity are obtained, can be varied.

Various striking pads ranging from wood to various combinations of steel springs are used between the striker and the anchor to encourage a near elastic impact and thus a more controlled energy output.

SENSING AND RECORDING EQUIPMENT

In each borehole, one velocity transducer is used to measure the velocity/time history in the vertical direction. Each transducer attached to a holder is lowered into the borehole wall. An air-inflated sensor holder is used to couple the sensor to the sides of the borehole.

Typical data obtained in 3 holes at about 5-foot spacing are shown in Fig. 2. The distance between each vertical line represents 5 ms of time. The velocity of the shear wave is about 1000 fps and the shear strain about 5×10^{-5}%.

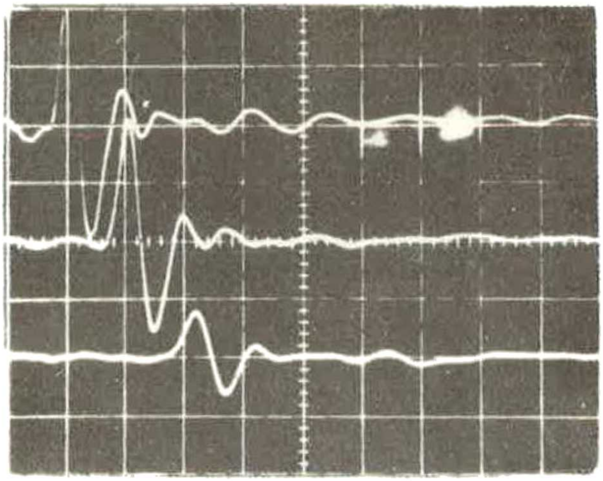


Fig. 2 Typical Soil Velocity Data

Chairman Prof. V.S.Eristov
Thank you very much Mr. Wilson. for your contribution. The next will be Mr.Uriel

A.O.Uriel (Spain)

The possibility of cracking in the clay core of an earth-rock dam may be analysed by means of the Finite Element Method. In particular, solutions are available in the technical literature in which the possibility of generation of vertical tension cracks in the vicinity of the abutments, as a consequence of their steepness, is investigated. But, at least as far as I know, clay is assumed to be isotropic and elastic, which may be greatly in error and may lead to conclusions on the unsafe side.

In the situation of full reservoir, the state of stresses and strains in the extension zones of the core (Fig.1) is mainly governed by a compression in the "x" and "z" directions, and an extension (in terms of strains rather than stresses) in the "y" direction. Due to the fact that the modulus of elasticity in the direction of the applied compressions and in the direction of the extension should not be the same, the clay behaves anisotropically. In the last International Congress on Large Dams, held in Madrid in June, Prof. De Mello showed the results of carefully conducted tests which confirm that the elasticity moduli of a clay in extension and in compression are different.

Let us assume that the induced anisotropy is of the transverse type, with its axis of symmetry in the direction of the extension. In this case, there are five elastic constants (E'_x, E'_y, μ_{xx}, μ'_{xy} and G_{xy}), and two main parameters can be defined to express the degree of anisotropy, $n'=E'_x/E'_y$ and $r'=(1-\mu'_{xx}-\mu'_{xy})/(n'-2\mu'_{xy})$. The first one is the ratio of the elasticity modulus in the compression direction to the extension direction. The second one is the compressibility ratio under uniaxial stress in the principal directions.

Cracking is a complex phenomenon associated with an increase in volume, which produces a loss of cohesion. So, if the volume of soil remains unchanged during the deformation process or, even, decreases, cracking will not take place.

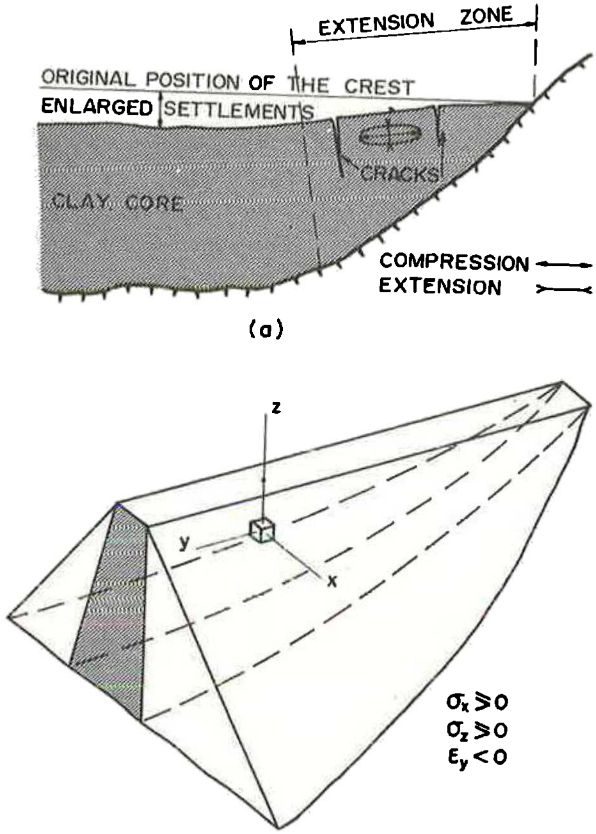


FIG. 1

The expression for the decrease in volume due to the compressions σ'_x and σ'_z and the extension ε_y is:

$$d_s = 3\varepsilon_{oct} = \frac{(n'-n'\mu'_{xx}-2\mu'^2_{xy})(\sigma'_x+\sigma'_z)+}{n'E'_x}$$

$$\frac{+(n'-2\mu'_{xy})E'_x\varepsilon_y}{n'E'_x}$$

The elastic parameters cannot take any arbitrary values, but there is a limit to their variation as a result of the strain energy which must be positive. Some of the restrictions are:

$(n'-n'\mu'_{xx}-2\mu'^2_{xy}) \geqslant 0$

$E'_x > 0$
$n' \geqslant 0$

Besides, $\sigma'_x \geqslant 0$, $\sigma'_z \geqslant 0$ and $\varepsilon_y < 0$, by definition. Should the term $(n'-2\mu'_{xy})$ be negative, the volume change will be positive, in other words, volume decreases. This is equivalent to $r' \leqslant 0.5$, corresponding to those soils which dilate under uniaxial compression in the "y" direction.

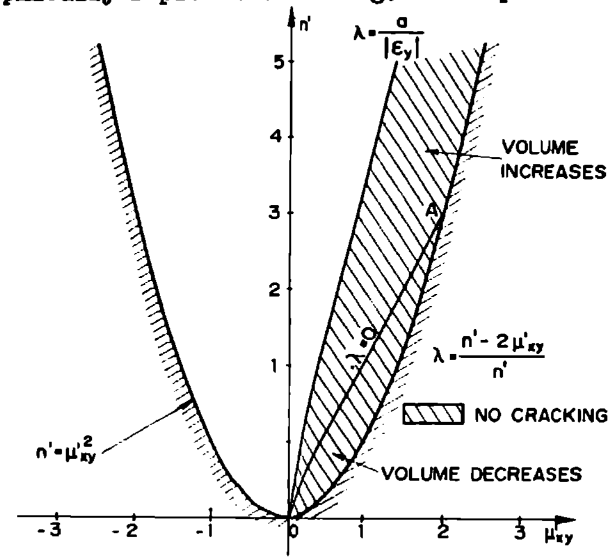
The worst conditions are in the crest, where σ'_x and σ'_z are zero and ε_y reaches its maximum value. The expression for the volume change reduces to:

$$d_s = \frac{n'-2\mu'_{xy}}{n'} \varepsilon_y$$

If a certain amount of increase in volume "a" can be allowed without generation of cracks, the condition for no cracking will be:

$$\lambda = \frac{n'-2\mu'_{xy}}{n'} \leqslant \frac{a}{|\varepsilon_y|}$$

The criterion for cracking has been graphically represented in Fig.2. The parabola



CRACKING POTENTIAL IN TERMS OF THE ELASTIC PARAMETERS

is the limit to the variation of the parameters n' and μ'_{xy}. All those soils with the same value of the parameter λ are represented by a straight line passing through point B. Below the line $\lambda = 0$, there is a decrease in volume. Between this line and the line $\lambda = a/|\varepsilon_y|$, there is an increase in volume, but less than allowed. So, for those clays lying in the shaded area, cracking under the conditions mentioned is not possible.

If isotropy is assumed in a Finite Element analysis, then n' is equal to one. Taking a common value of 0.3 for the Poisson's ratio the parameter λ is equal to 0.4. In this case, the condition for no cracking is $\varepsilon_y \leqslant 2.5a$.

But really, n' is several times greater than one, and with a realistic value of the Poisson's ratio μ'_{xy}, the parameter λ approaches unity. The limitation for the maximum strain ε_y for no cracking is then restricted to $\varepsilon_y \leqslant a$.

In conclusion, the induced anisotropy of the clay of an impervious core must be taken into account when analysing the possibility of cracking. Otherwise, safety against generation of tension cracks is not warranted.

Chairman V.S.Eristov.
Thank you Mr. Uriel. Prof.Lowe will you please

John Lowe,III(USA)

Consolidated Undrained Tests on Coarse Grained Core Material

I wish to bring to the attention of the Congress what I believe are the first consolidated undrained 'R' type triaxial tests performed on large diameter specimens. These are tests performed during the last several years on the core material for Tarbela Dam, Pakistan.

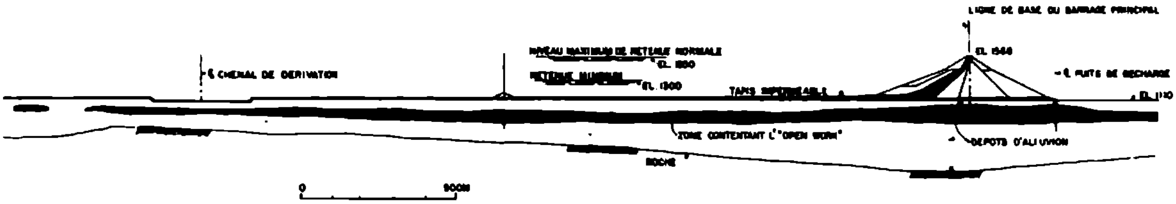
The core for Tarbela Dam is inclined upstream and connects with an upstream impervious blanket as shown in Figure 1. The great depth of alluvium in the foundation of the dam, up to 200m, precluded the use of a positive cut-off to bedrock.

The core material consists of a blend of sand, gravel and cobbles with sandy silt. The grain size of the blended material is shown in Figure 2. The maximum size of particle in the blend is 15 cm. The average percentage passing the No.200 sieve is 30%.

Tests were performed on specimens 0.61m in diameter and 1.52m in height. A photograph of a specimen after test is shown in Figure 3. Three triaxial machines were available for testing. Chamber pressures up to 35 atmospheres could be used. Generally back pressures of 7 atmospheres were used during consolidated undrained testing.

The pore pressure and deviator stress measurements made during three tests are shown on Figure.4. Test 1 had a chamber pressure of 30 lb/in^2. Test 2, 60 lb/in2 and test 3, 105 lb/in^2. Negative pore pressure developed in Test 1 and positive pore pressures in Tests 2 and 3. The deviator stress reached a maximum value at 5% strain and experienced only slight increase, if any, until 16% strain.

The Mohr envelopes for total normal stresses above back-pressure and effective normal stresses are shown in Figure 5. The envelope



BARRAGE DE TARBELA

COUPE DU BARRAGE PRINCIPAL EN REMBLAI ET DU TAPIS IMPERMEABLE

Fig.1.

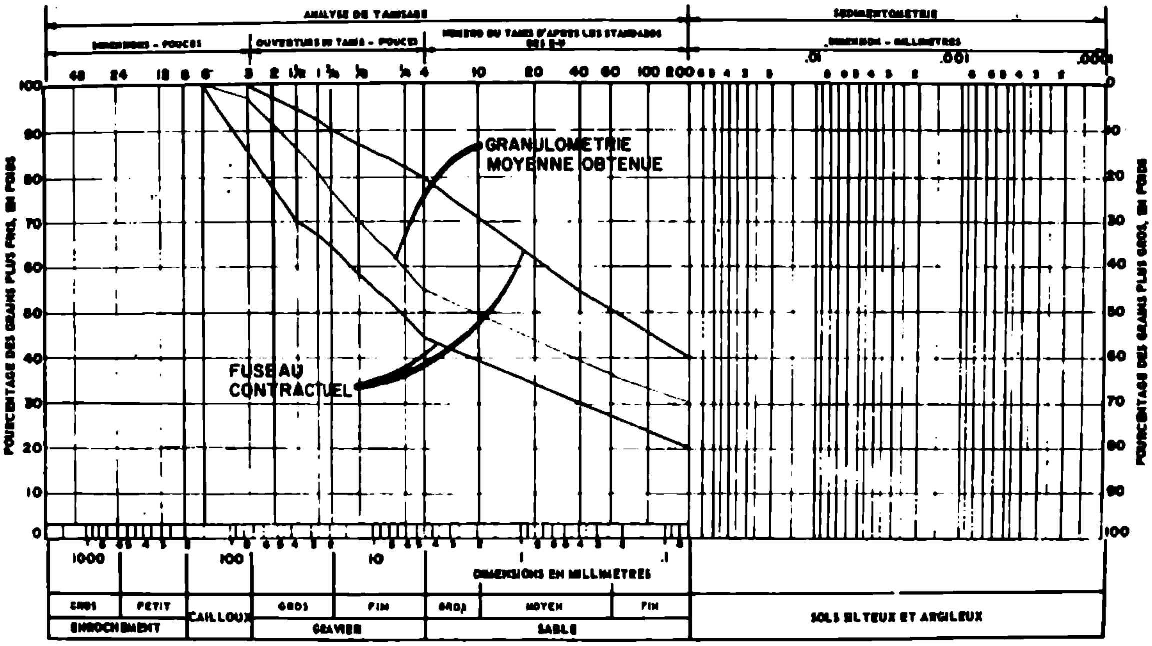


Fig.2

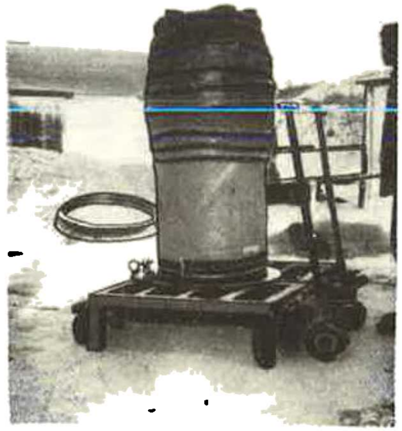


Fig.3.

of the effective stress circles passes through the origin and gives an effective angle of 39°. Only slight degradation of the material took place during compaction and testing as shown on Figure 6.

Tests with 20% and 40% passing the No.200 sieve were performed also. In addition, triaxial tests were performed on 'as compacted' specimens unsaturated, unconsolidated undrained or 'Q' tests as well as consolidated drained 's' tests.

Progress photographs of the construction of the Tarbela embankment dam are shown in Figure 7 through 11. The view is from the right abutment looking toward the left.

Although about only 80% complete, Tarbela Dam is now the largest dam in the world.

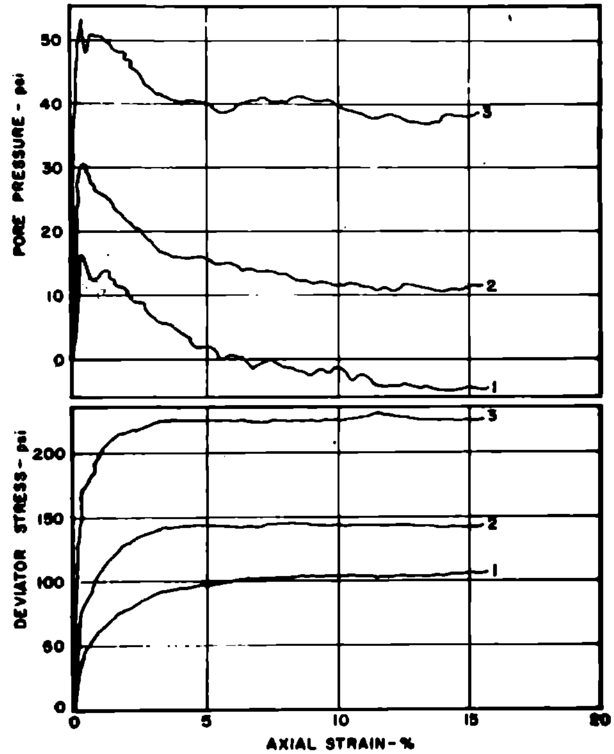


Fig.4.

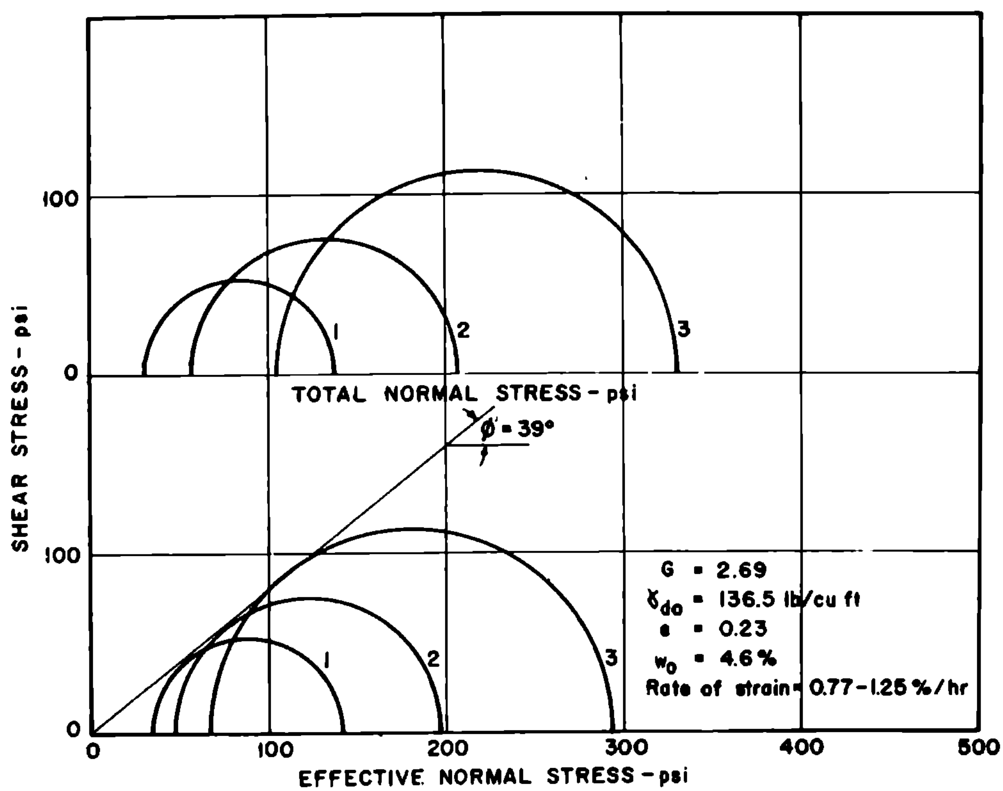


Fig.5

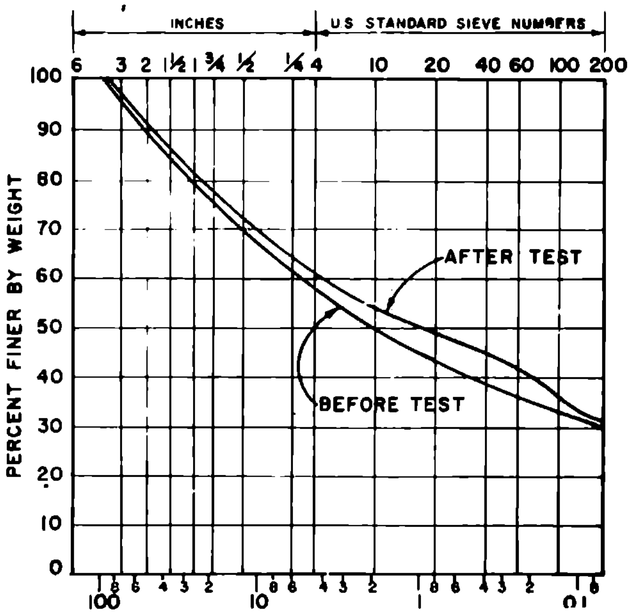


Fig. 6

Fig. 7

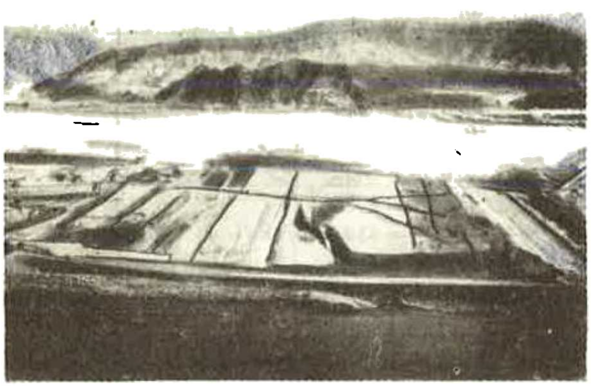


Fig. 8

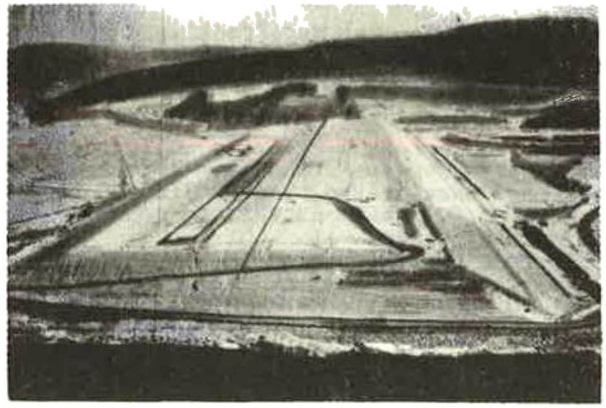


Fig. 9

Fig. 10

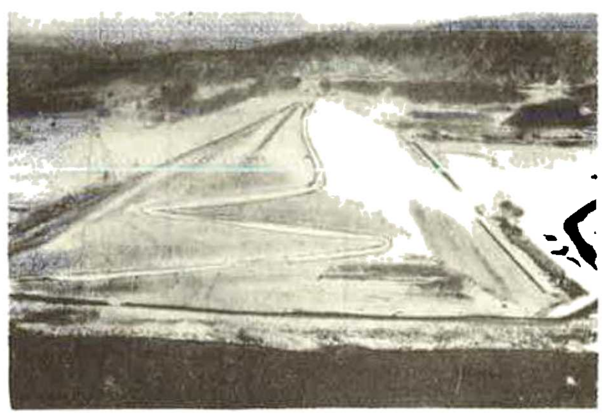


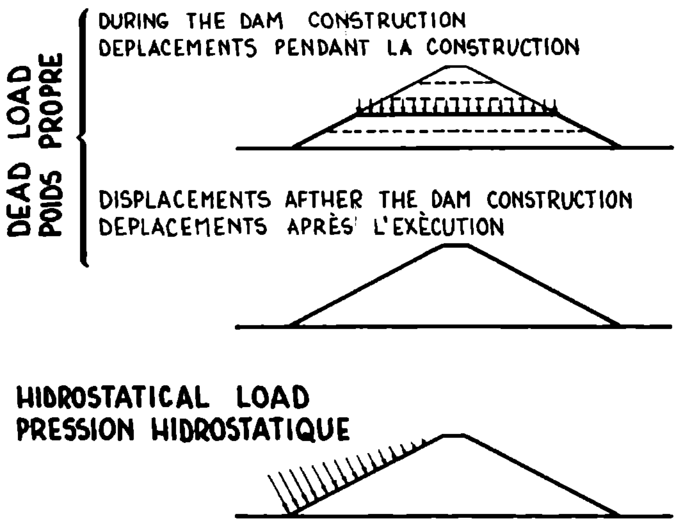
Fig. II

Chairman V.S.Eristov
Thank you Prof. Lowe. The next will be Mr.
Comsa from Romania

Comsa, R.S. (Romania)

Dans une discussion publiee dans cette session speciale nous avons souligne l'utilite de l'emploi des coefficients non-dimensionaux,calcules sur un modele d'elements finis, pour l'analyse et la generalisation des resultats des mesures de deplacement.

Les deplacements peuvent etre exprimes a l'aide des coefficients non-dimensionaux utilisant des schemas corespondantes aux charges appliquees (Figure 1.)



$$U_a = \frac{\gamma_b H^2}{E} \cdot C_a$$

$$U_b = \frac{\gamma_b \cdot H^2}{E} \cdot \frac{\Delta E}{E - \Delta E} \cdot C_b$$

$$U_c = \frac{\gamma_a H^2}{E} \cdot \frac{E}{E - \Delta E} \cdot C_c$$

$$U = \frac{\gamma_b \cdot H^2}{E} \cdot \left[C_a + \frac{\Delta E/E}{1 + \Delta E/E} \cdot C_b + \frac{\gamma_a/\gamma_b}{1 + \Delta E/E} \cdot C_c \right]$$

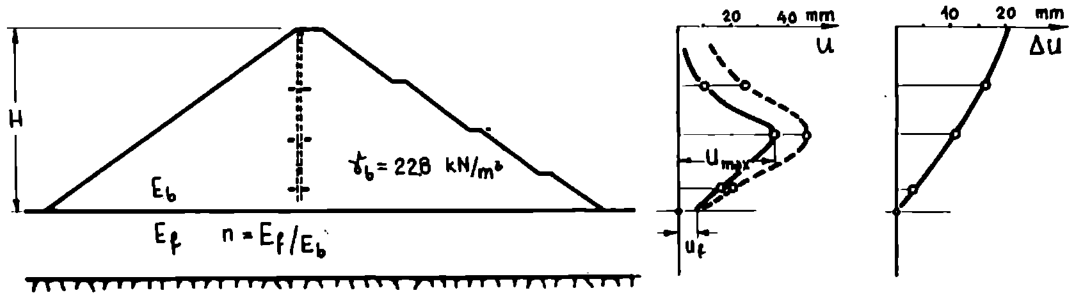
Fig.1

Par le suite on presentera deux exemples d'application de cette tehnique de calcul de deplacements.

Dans le premier exemple (Figure 2) les deplacements mesures au niveau de la fondation par Kazda, 1971, ont ete utilises pour le calcul de module d'elasticite de la fondation. Le rapport, n, entre les modules de la fondation et du barrage a ete obtenu a partir du rapport entre le deplacement maximal dans le barrage et le deplacement de la fondation.

Les deplacements rheologiques (la difference entre les mesures faites a le fin de la construction du barrage et celles faites quelques mois apres) on ete equivales a une reduction du module d'elasticite en temps.

On a interprete, dans le deuxiem exemple (Figure 3), les mesures de deplacements du talus amonte de barrage de Salt Springs (Steele, Cooke, 1958). La position du maximum des valeurs mesurees a servi a etablir le rapport n, et par le meme le module d'elasticite du barrage. Les calculs faites a l'aide de la metode des elements finis, en utilisant cettes caracteristiques, relevent une tres bonne coincidence avec les mesures, tant pour in temps court que pour une longue periode de temps.

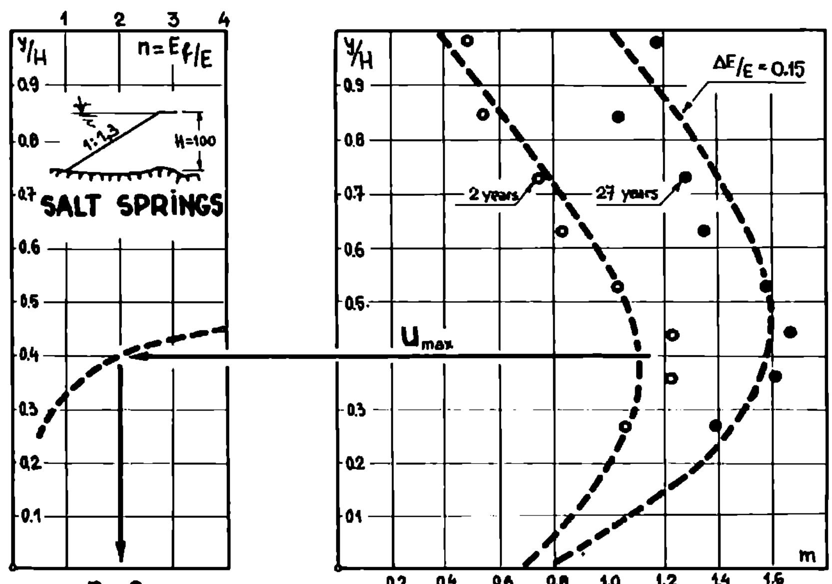


$$u_f \longrightarrow E_f = \frac{C_a'}{u_f} \cdot \gamma_b H^2 = 14\,000\ \mathrm{N/cm^2}$$

$$\frac{u_{max}}{u_f} \simeq 6 \rightarrow n > 10 \rightarrow E_b = \frac{C_a}{u_{max}} \cdot \gamma_b \cdot H^2 = 1350\ \mathrm{N/cm^2}$$

$$\frac{\Delta E}{E} = \frac{1}{\frac{C_b}{\Delta u} \cdot \frac{\gamma_b \cdot H^2}{E} + 1} = 0.198$$

Fig.2



$n = 2$

$\downarrow$

C_c

$\downarrow$

$$E = \frac{C_c}{u} \cdot \gamma H^2 = 4000\ \mathrm{N/cm^2} \longrightarrow$$

o MEASUREMENTS

--- FE ANALYSIS

Fig.3

References.

Kazda, I. (1971). L'application de la metode des elements finis au calcul de deformations des ramblees (en tcheque). Vodni hospodarstvi, nr.7

Steele, I.C. et Cooke, J.B. (1958). Salt Springs and Lower Bear River concrete face dams. Proc. ASCE, PC 4.

Chairman V.S.Eristov
Thank you Mr.Comsa.Mrs.Lipovetskaya will you please

T.F.Lipovetskaya (USSR)

Presented are the results of the investigations carried out by a group of scientists at the B.E.Vedeneev All-Union Research Institute of Hydraulic Engineering for the impoundment of the cooling pond of the Uglegorsk thermal power plant. The earth dam of the Uglegorsk thermal power plant is a uniform earth fill embankment of loam constructed by a layer-by-layer technique. The length of the dam is 2.6 km, its maximum height 32m, the crest width 11m, the slope angle 1:3. The downstream shoulder of the dam contains a flat drainage zone connected to a drainage shell. The dam foundation is composed of quaternary deposits overlying the sedimentary bed rock of the Early Permian formation. The weakest alluvial deposits are loam layers 4 and 5 with consistencies ranging from liquid-plastic to solid-plastic. Their maximum layer thickness is about 12m.

From the engineering geological examination the shearing resistance of loam in layers 4 and 5 is characterized by the internal friction angle φ = 19-20° and the unit cohesion C= 0.20 kgf/cm². Based on the above parameters a dam was designed of a reduced cross-section incorporating a drainage shell in its downstream shoulder.

However after stripping the upper layers of the foundation pit the loam in layers 4 and 5 in the channel section proved to be moist, unconsolidated and nearly impermeable, which would make erection of the earth embankment impossible. The following characteristics were obtained by in situ laboratory tests of the loam: moisture content up to 43%, unit weight 1.48 t/m³, saturation coefficient near to unity and consistence ranging from liquid to plastic. The shearing resistance of undisturbed loam was obtained from tests in a shearing apparatus by the quick shear method (φ =3°-7°, C=0.16-0.40 kgf/cm² for w=25-30% and γ=1.47 g/cm³) in a triaxial compression apparatus of a closed type (φ =0, C=0.20 kgf/cm² for w=29% and γ=1.49g/cm³). Slope stability calculations for the above characteristics of the slope stability revealed that the safety coefficient for the dam was well below unity.

Experiment also showed that consolidation in the dam foundation would be rather slow and consequently the stability of the dam by the moment of its completion would be inadequate.

To improve the stability of the dam one had either to flatten the slopes or to provide for accelerated consolidation of the foundation soil, as was the case with the Sauri dam in Japan where the foundation was pierced by a thousand sand piles. All these measures would have led to a considerable increase in construction costs.

Fortunately an additional engineering geological survey revealed the presence of sandy and loamy lenses in layers 4 and 5 facilitating seepage. Thus it was decided to construct a dam with a reduced cross section. Observations on the consolidation of the foundation soil were undertaken to assess the state of the soil and the dam stability at different stages of construction. In 1971-1972 when the dam was erected to a height of 15m, 56 holes were bored in two measuring section in the channel zone of the dam to install 13 precison piezometers and 36 piezodynamometers and to take soil samples from the embankment and the foundation. Testing of samples taken during the construction stage showed that due to the effect of the weight of the already constructed section of the dam the soils in the foundation was compacted to a certain extent and their shear resistance increased. The average moisture content of the soil decreased to 24%. The dry weight of soil increased up to 1.57 g/cm³.

According to a quick shear tests, the shearing resistance of soil in layers 4 and 5 was φ=13° and C= 0.2-0.3 kgf/cm², respectively. The triaxial tests showed the values φ=5° and C=0.3 kg/cm2. As the conditions of the consolidation of soil under the upstream and downstream dam shoulders were different because of the plane drainage under the downstream fill, different values of strength characteristics of layers 4 and 5 were adopted in the stability calculations of the upstream and the downstream fill of the dam, higher values being adopted for the downstream portion.

The slope stability was calculated by the inclined force method of R.R.Chugaev and by the general method of A.L.Mozhevitinov. Calculation results have shown that for the upstream slope of the dam K_s=1.40-1.44 and for the downstream K_s=1.22-1.31, with the standard margin of safety K_s= 1.30. The analysis, design procedures and in situ investigations of the earth dam in complex engineering-geological conditions have proved that dams with a reduced cross-section can be constructed without special measures for speeding-up the consolidation of the foundation soil.

Chairman V.S.Eristov.
Thank you Mrs.Lipovetskaya for your contribution. Now I pass the word to Mr.Habib

P.Habib (France)

Apres la premiere mise en eau d'un barrage en terre homogene, on voit parfois apparaitre des fissures. Ces desordres exterieurs sont faciles a surveiller mais il peut s'en produire dans le corps de l'ouvrage dans des zones ou l'observation est pratiquement impossible. Dans certains cas ce phenomene peut s'expliquer par le mecanisme suivant: lors de la construction le barrage tasse sous l'action du poids propre; lorsque les infiltrations d'eau envahissent le barrage les parties humides deviennent plus compressibles et tassent a nouveau sous l'effet du poids propre. On peut donc calculer les contraintes

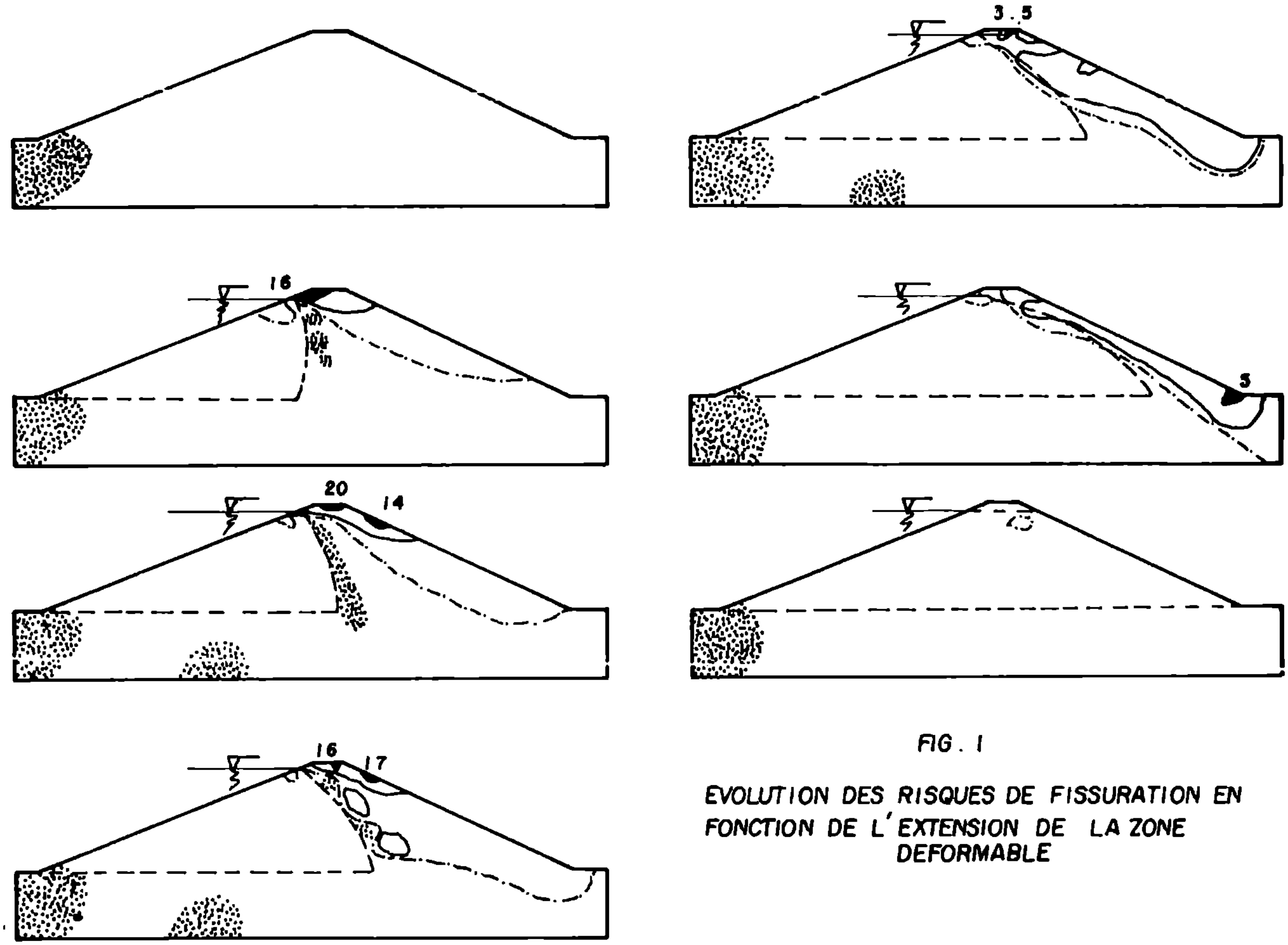


FIG. 1

EVOLUTION DES RISQUES DE FISSURATION EN FONCTION DE L'EXTENSION DE LA ZONE DEFORMABLE

dans l'ouvrage en supposant le barrage heterogene, avec un module d'elasticite plus faible dans la zone humide que dans la zone seche, et soumis au poids des terres et aux forces dues a l'eau.

Le calcul a ete fait par la methode des elements finis et on a compare les contraintes avec le critere de rupture du sol pour determiner les zones de rupture commençante. On peut definir le risque de rupture par le quotient du rayon du cercle de Mohr divise par la distance du centre du cercle de Mohr a la courbe intrinseque;on voit que lorsque ce rapport est inferieur a l'unite on est dans le domaine de la stabilite;on voit aussi que le risque de rupture est d'autant plus important que ce rapport est grand.Le calcul elastique n'est evidemment acceptable que si les zones de rupture sont peu etendues.

La rupture peut etre:

- ductile lorsque la contrainte moyenne est grande,

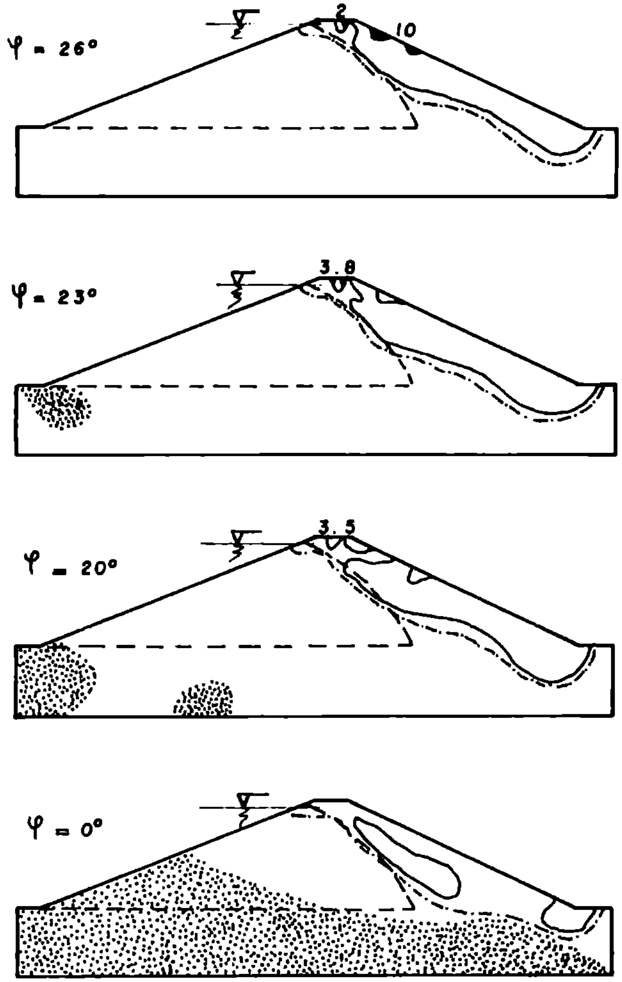


FIG. 2

EVOLUTION DES RISQUES DE FISSURATION EN FONCTION DU FROTTEMENT INTERNE

- plastique avec lignes de glissement pour des contraintes moyennes moderees,
- avec fissuration si l'une, au moins, des contraintes principales est une traction

Ceci permet de preciser le risque de fissuration

En faisant ainsi le calcul pour une coupe de barrage simple nous avons vu apparaitre trois points de danger de fissuration:

- sur la crete du barrage
- au tiers superieur du parement aval
- dans certains cas, dans le corps du barrage en avant du front d'invasion

La figure 1 represente l'evolution des courbes de risque de fissuration avec l'avancement de la zone d'infiltration.

La figure 2 represente l'evolution des courbes de risque de fissuration avec la variation de l'angle de frottement interne.

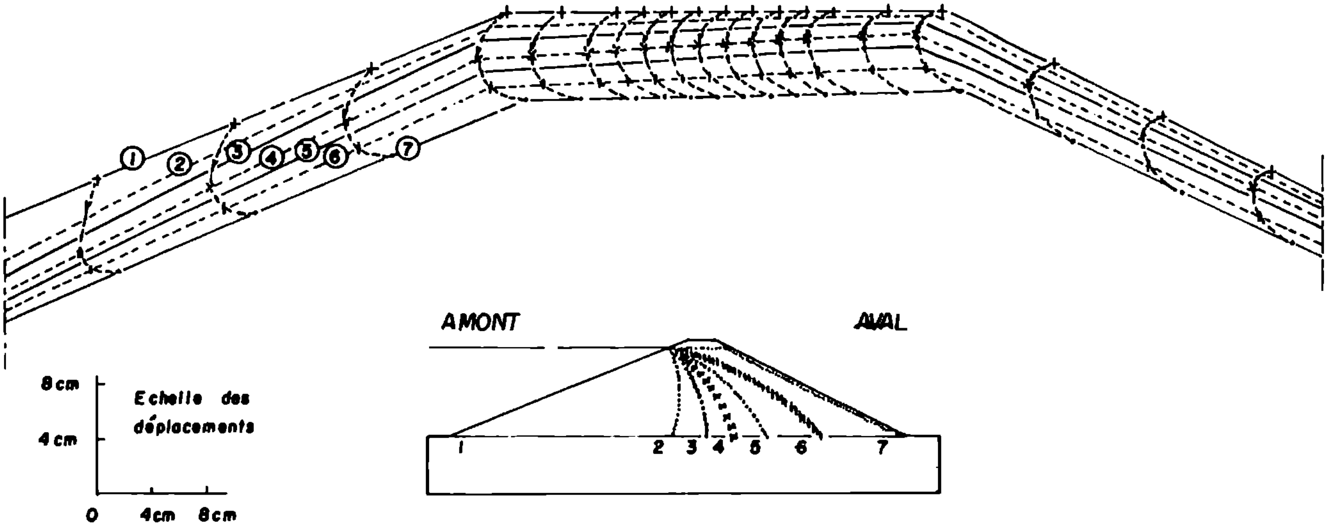


FIGURE 3

Evolution des déplacements au sommet du barrage avec l'extension de la zone mouillée présentant une déformabilité plus grande

Le calcul par la methode des elements finis permet d'obtenir les deformations. La figure 3 indique le deplacement de la crete de l'ouvrage avec l'avancement de la zone d'infiltration. On voit notamment que le mouvement a commence vers l'amont,ce qui a effectivement ete observe sur un ouvrage reel.

Chairman V.S.Eristov.
Thank you Mr. Habib . The next will be Mr. Perlea from Romania

Vlad D.Perlea (Romania)

In the case of rock-fill dams with sloping or central core,usually it is assumed that the failure can occur along a slip surface consisting, in the cross section, of two or three straight lines. (fig.1).

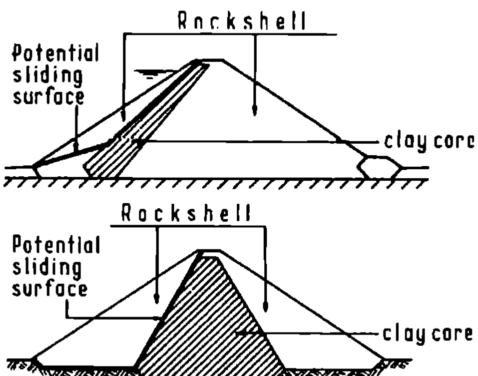
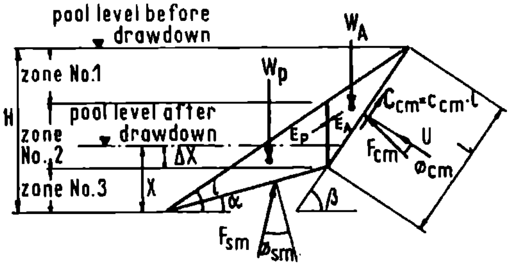
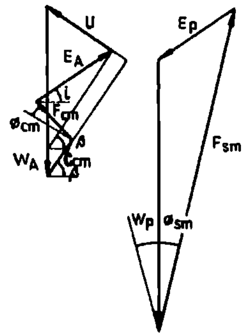


Fig.1

For solving the stability problem by limit equilibrium method,the potential failure mass is broken up into two or three wedges. A trial and error procedure is used,assuming various safety factors until the force polygon closes. Of course,the critical potential failure surface must be determined by trials too (i.e. the angle α)(fig.2).

Stability analysis can be greatly facilitated by the use of stability charts. Such charts were prepared for the following three hypotheses: (1) the end of construction, (2) steady seepage with partial pool, and (3) sudden drawdown. These charts were prepared by computer calculations, based upon a relationship (fig.2) rather complicate and depending on the relative position of failure surface and water level in the pool, where the coefficients A,B,C,S and E depend only on the dam geometrical characteristics, and not on the position of presumed slide surface. For establishing the relationship some experimental and theoretical interior works were utilized (Ciugaev, 1964, Seed and





$C_{cm}=\frac{C_c}{F}$ $\quad C_c=c_c\cdot l$

$\tan\phi_{cm}=(\tan\phi'_c)/F$

$\tan\phi_{sm}=(\tan\phi'_s)/F$

$U=V'_A(\gamma-\gamma')\cdot\cos\beta$

where :

V'_A-is the part of the active wedge volume situated between the levels before and after drawdown

γ-unit weight after drawdown

γ'-submerged unit weight

from the condition $E_A=E_p$ it results :

$$\frac{c_c}{F\gamma H}=\frac{\sin i}{C}\cdot\frac{\sin(\beta-\alpha)}{\gamma H^2\sin(i-\alpha)}\,(W_A\cdot B+W_p\cdot S+U\cdot E)$$

in which (for zone No.1) :

$$\frac{W_A}{\gamma H^2}\cdot\frac{\sin(\beta-\alpha)}{\sin(i-\alpha)}=\frac{A}{2}\cdot\frac{\cos\beta\cdot\sin(i-\alpha)}{\cos\beta\cdot\sin(\beta-\alpha)}\left(\frac{\cos\beta}{\cos i}+A\,\frac{\gamma'}{\gamma}\right)-\frac{\Delta X}{H}\cdot\frac{A}{\sin\beta}\cdot\frac{\gamma-\gamma'}{\gamma}\left[\cos\beta-\frac{1}{2}\,\frac{\Delta X}{H}\cdot\frac{\cos i\cdot\sin(\beta-\alpha)}{\sin(i-\alpha)}\right]$$

$$\frac{W_p}{\gamma H^2}\cdot\frac{\sin(\beta-\alpha)}{\sin(i-\alpha)}=\frac{A^2}{2}\cdot\frac{\gamma'}{\gamma}\cdot\frac{\cos i\cdot\cos\alpha}{\sin(\beta-\alpha)}$$

$$\frac{U}{\gamma H^2}\cdot\frac{\sin(\beta-\alpha)}{\sin(i-\alpha)}=\frac{A^2}{2}\cdot\frac{\cos^2\beta}{\sin\beta}\cdot\frac{\gamma-\gamma'}{\gamma}\left[\frac{\sin(i-\alpha)\cdot\cos\beta}{\sin(\beta-\alpha)\cdot\cos i}-2\,\frac{\Delta X}{H}+\left(\frac{\Delta X}{H}\right)^2\cdot\frac{\sin(\beta-\alpha)}{\sin(i-\alpha)}\right]$$

(and similar for zones No.2 and 3),where :

$$A=\frac{\sin\beta}{\sin i}-\frac{\cos\beta}{\cos i}\qquad B=\sin i-\cos i\cdot D\qquad C=\cos(\beta-i)-\sin(\beta-i)\cdot D$$

$$D=\frac{(\tan\phi_c)/F-\tan(\beta-i)}{1+\tan(\beta-i)\cdot(\tan\phi_c)/F}\qquad E=\sin(\beta-i)+\cos(\beta-i)\cdot D\qquad S=\sin i-\cos i\cdot\frac{\tan(i-\alpha)+(\tan\phi_s)/F}{1-\tan(i-\alpha)\cdot(\tan\phi_s)/F}$$

Fig.2

Sultan, 1967, Sultan and Seed, 1967, Thurner, 1970).

Using the charts (e.g.fig.3), one can obtain the necessary cohesion for the core material (c_c), if all the following parameters are given: H-the heigh of the dam; F-the factor of safety; β -the slope of the core; ϕ_{cm}-the mobilized internal friction in core ($\tan\phi_{cm}=\tan\phi_c/F$); ϕ_{sm}- the mobilized internal friction in shale ($\tan\phi_{sm}=\tan\phi_{sm}/F$); i-the angle of external slope which is equal or less than the mobilized internal friction in the cohessionless material. Also one can obtain the critical slip surface (i.e.the critical α angle) and the critical position of the water level in the pool after drawdown (i.e.the critical X to H ratio).

It is interesting to note that for the two cases shown in fig.4, the clasical Swedish method with circular failure surfaces gives a factor of safety which is smaller than the one obtained by the wedge method. Thus, the factors of safety calculated by Swedish method for different values of the parameters corresponding to a factor of safety of 1.5 obtained by wedge method, were smaller than the above mentioned value. These factors of safety are shown in fig.4 next to each corresponding point.

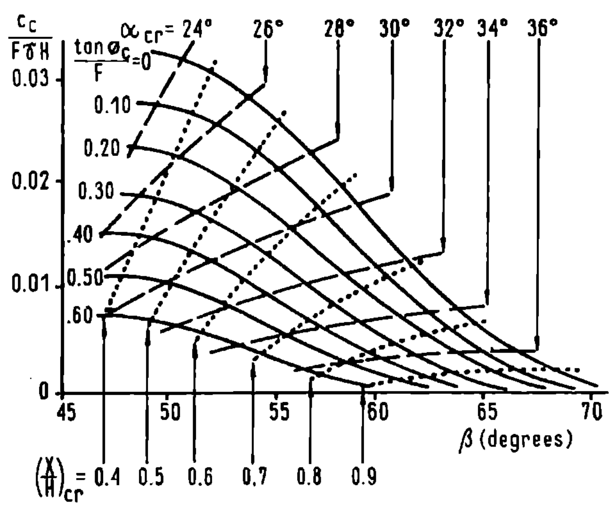


Fig.3

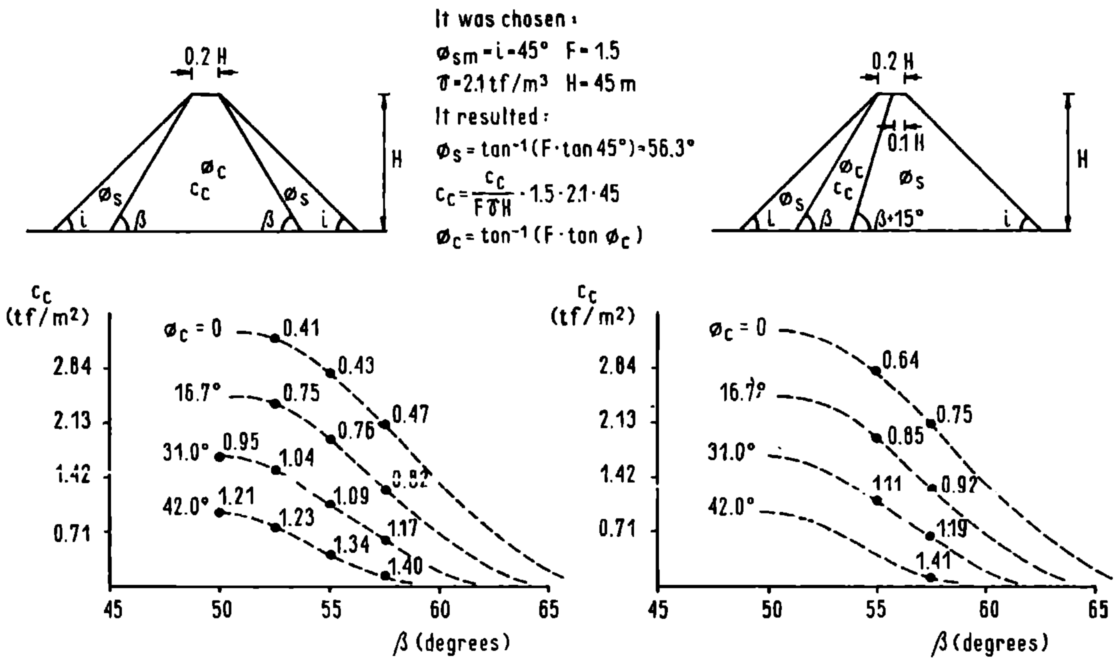

0.2 H
H
Øc
cc
Øs
i
β
It was chosen:
Øsm = i = 45° F = 1.5
γ = 2.1 tf/m³ H = 45 m
It resulted:
Øs = tan⁻¹(F·tan 45°) = 56.3°
cc = cc/(FγH) · 1.5 · 2.1 · 45
Øc = tan⁻¹(F·tan Øc)
0.1 H
β+15°
cc
(tf/m²)
2.84
2.13
1.42
0.71
Øc = 0
16.7°
31.0°
42.0°
0.41
0.43
0.47
0.75
0.76
0.82
0.95
1.04
1.09
1.17
1.21
1.23
1.34
1.40
0.64
0.75
0.85
0.92
1.11
1.19
1.41
45
50
55
60
65
β (degrees)

Fig.4

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Chairman Prof.V.S.Eristov.
Thank you Mr. Perlea for your discussion. Now we'd like to listen to Mr. Hosking from Australia

Hosking A.D. (Australia)

Four years ago at the Seventh Conference in Mexico City the authors of the State of the Art paper on Earth and Rockfill Dams remarked as follows:
"At the present time, very little quantitative data exists concerning the development of strains and cracking in embankments".

It may be of interest to them and to this Conference, to know the results of strain measurements taken since then in the crest of Talbingo Dam, an earth-rock structure over 160 metres high, built as part of the Snowy Mountains Scheme in Australia in the years 1968-1970.

It is well known that experience at most other high dams of this type,Round Butte, Cougar, El Infiernillo and Mattmark, to mention but a few, had shown that they were susceptible to transverse cracking at crest level, especially if there were any significant irregularities in the abutment profile.

Although there is no record of the failure of any large dam as a result of transverse cracking it is an obvious hazard which cannot be neglected. Every conventional defensive measure against cracking had been incorporated in the design of Talbingo Dam, particularly as a marked irregularity, of geological origin, exists in the upper right abutment. However at the time of the design in 1966-7 finite element methods of analysis were not generally available. When the dam was within a very few months of topping out, in late 1970, it was decided to install an extensometer system 122 metres long, over the irregularity to monitor the actual strains which would develop at this point. Figure 1 shows that the dam was already substantially instrumented and included horizontal movement installations on both upstream and

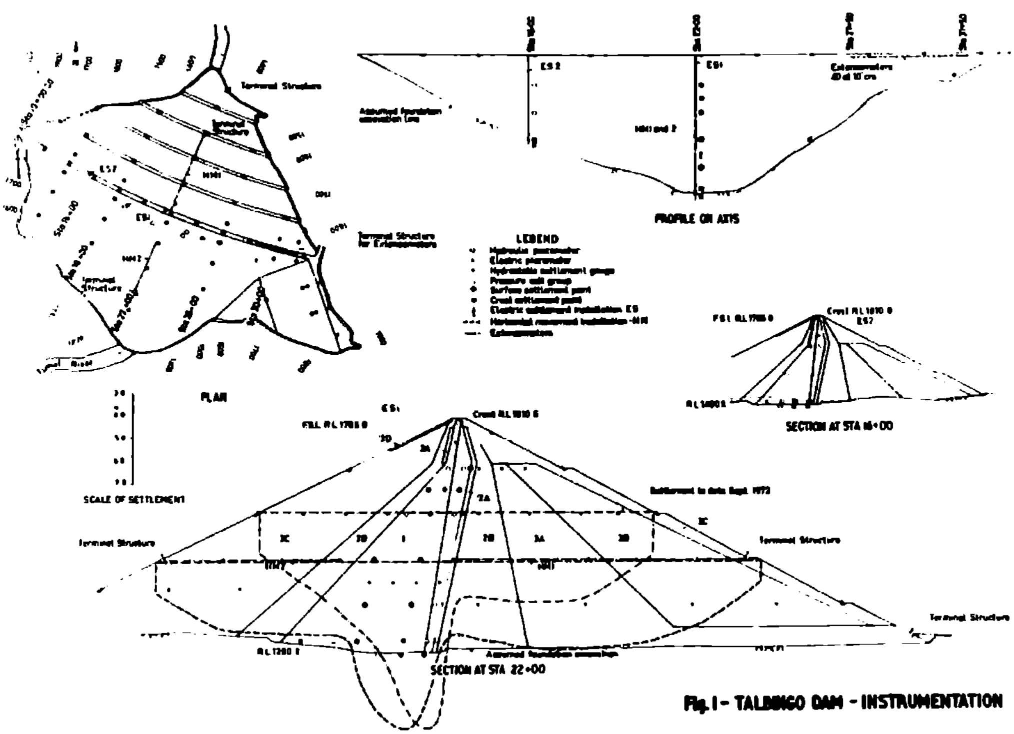


Fig.I - TALBINGO DAM - INSTRUMENTATION

downstream faces. However, use of this type of system in the crest would have required a shaft down to the terminal reading point and, moreover, the considerable distances between reading points (7 metres) may have caused a strain concentration to be overlooked.

A new system was quickly devised consisting of 40 telescoping sections each 3.04 metres long as shown in Figure 2. Each of these was connected to a crossarm. Movement between the crossarms actuates a rotating precision potentiometer by means of a bow string arrangement. The readout is by means of a Carlson bridge. There are two ball joints in each section to follow the curved crest of the dam and to ensure that moments do not affect the system. The installation is at a depth of 5 metres below the crest of the dam and it was placed entirely within a single day, a Sunday. It has operated faultlessly ever since.

Figure 3 shows how the system was arranged over the irregularity. It also shows, as expected, that small initial compressions developed at the ends of the system with tension in the middle. With time, and especially on first filling, these tensions developed considerably and reached a maximum of nearly 0.18 per cent before stabilising over the last year. This is almost exactly the same strain at which other experimenters have found cracking to start. However no cracks have been observed in the crest of Talbingo Dam yet.

A laboratory tensile test was performed on the Talbingo core material which is a weathered andesitic basalt containing an average of 15 per cent clay with a mean liquid limit of 36 per cent, a plasticity index of about 15 and with a Unified Classification of SC. The test specimens, which were 45 centimetres long and approximately 7.5 centimetres square, cracked at tensile strains in the range 0.22 to 0.51 per cent, depending upon moisture content and plasticity. These failure strains are thus slightly higher than those given in the 1969 State of the Art paper and elsewhere, and may reflect the fact that the tests were performed at a higher speed than most others. Talbingo Dam will continue to be monitored for cracks. Figure 4 shows

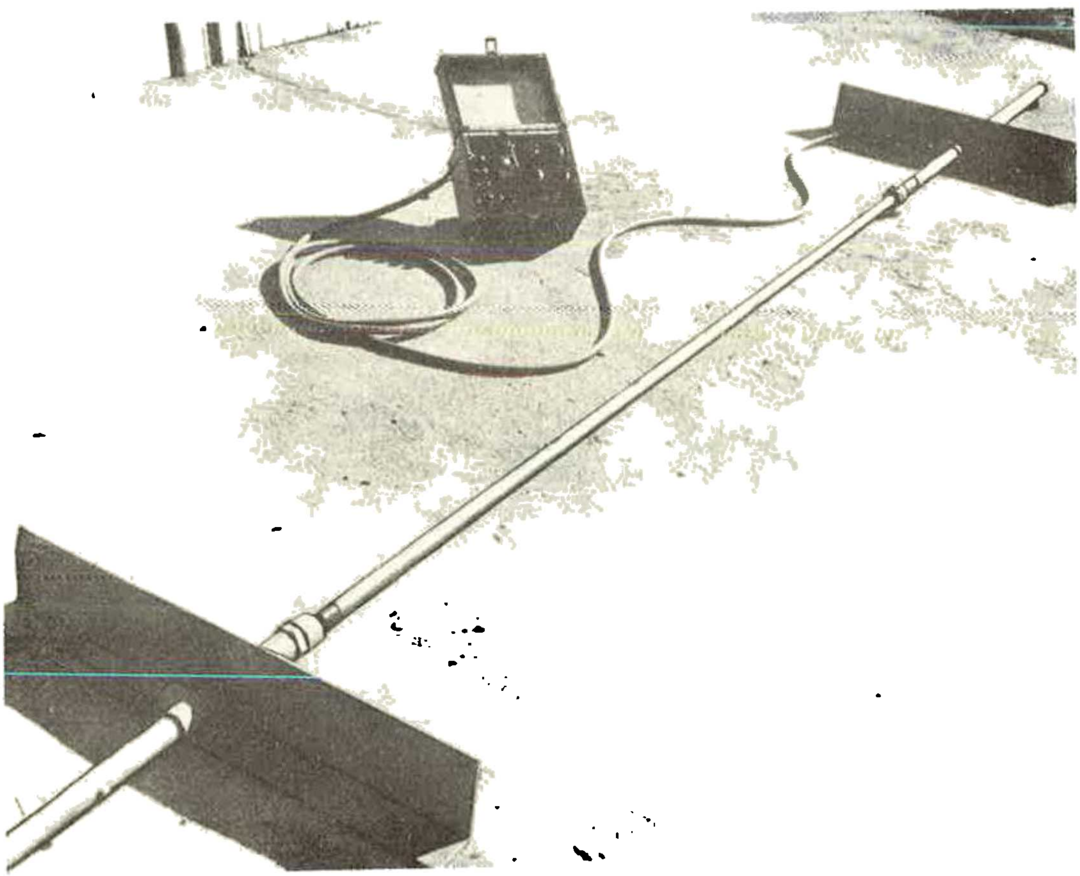


Figure 2

One of the 40 sections with two crossarms and the readout bridge. The telescoping unit is to the left of each crossarm.

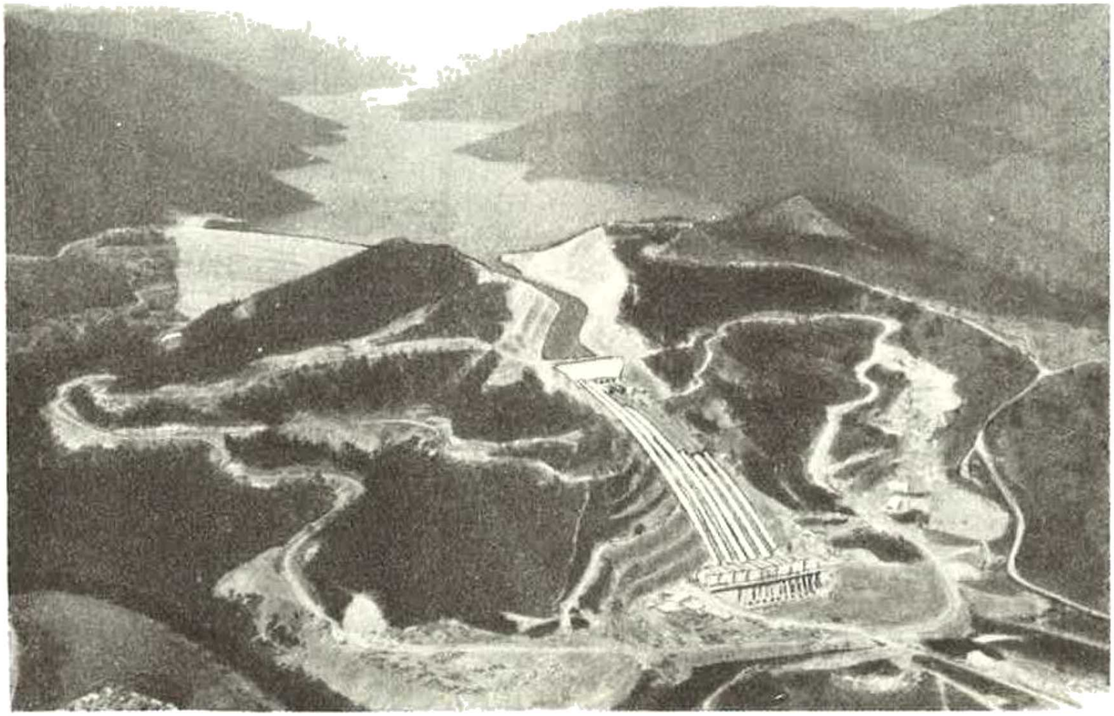


Figure 4

Tumut 3 Project nearing completion. Talbingo Dam, 180 metres, is on centre left.

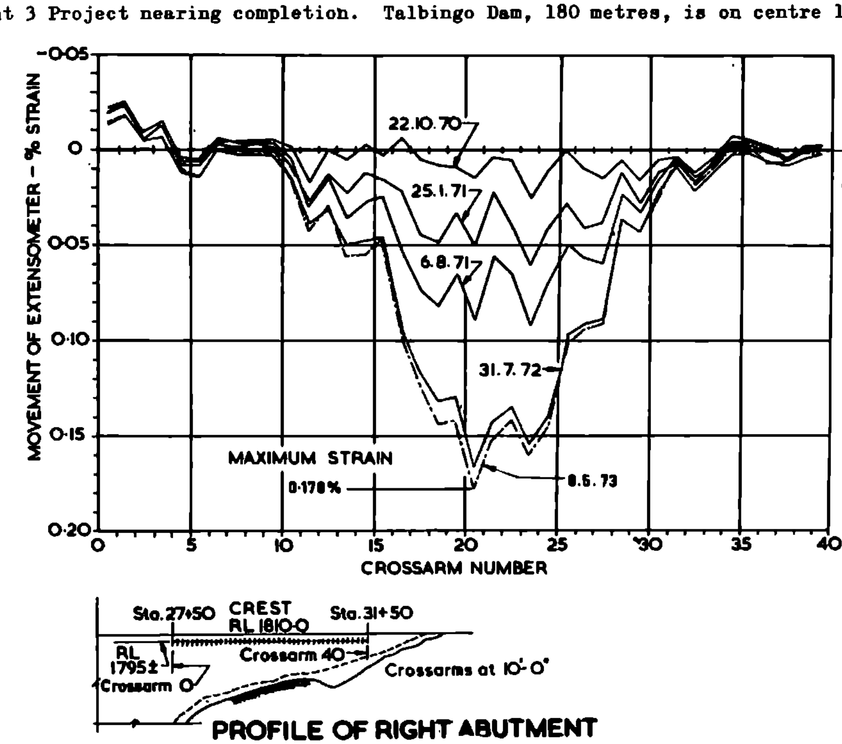


Fig. 3–TALBINGO DAM–EXTENSOMETERS

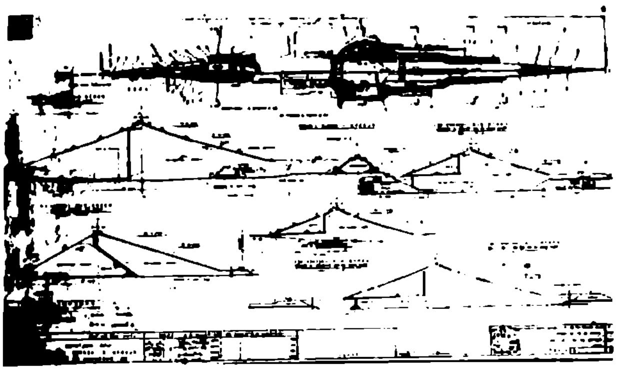
the completed dam, the headrace channel which was the main source of rockfill, and the nearly completed 1 500 MW power station.
Following this experience four such systems will be installed by the Snowy Mountains Engineering Corporation, for the State Rivers and Water Supply Commission of Victoria, in Dartmouth Dam. Construction of this dam which will be over 180 metres high is about to commence.

Chairman Prof. V.S.Eristov.
Thank you Mr. Hosking for your discussion
The next will be Mr.Hsu(Brasilia)

Serge J.C.Hsu (Brazil)

Mr.Chairman Colleagues!

I wish to present a brief information on the design and construction experience at Ilha Solteira dam on Parana River in Brazil, located some 700 km NW of the industrial city of São Paulo.
This hydroelectric project now nearing completion is the largest in Brazil, involving 3.200.000 kw when completed.
The first slide shows the plan and typical

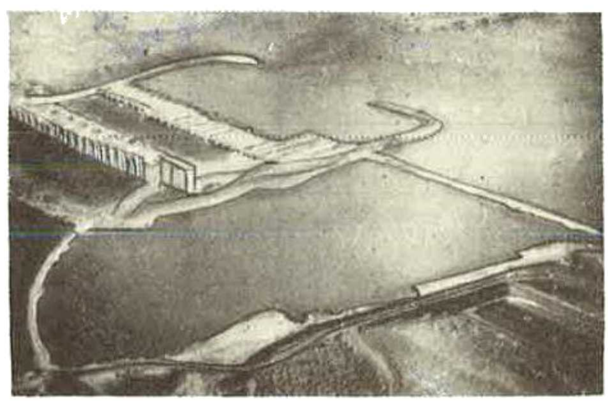


Sl.1.

sections of the earth dams with a maximum height of 65 meters in the river channel.
The right embankment is a homogeneous earth dam with built in sand drainage layers ("chimney drains"), typical of Brazilian design for sites where granular and sand materials are scarce.
The left embankment is an earth and rockfill dam.
The material used for the dam is a reddish brown, fine grained clayey sand or sandy clay of low plasticity and plots as CL type on the plasticity chart.
Known as lateritic soil, it is the mixture of colluvial and residual soils derived from sandstone and basalt. It is an ideal material for earth dam of this type as it has a high shear strength ($\phi = 30^{\circ}$), low permeability and is easily compacted to the required optimum density and moisture content.
The second slide shows the final river diversion made in May 1972.
The earthfill in the river channel was built between the two cofferdams "In dry", after

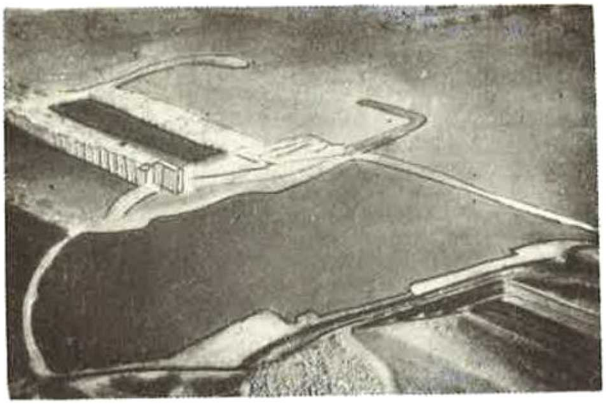
Sl. 2.

the final diversion of the river was done through 17 out of 20 partially completed power house units.
The 4 units were sealed off for erection work.
In order to permit early power generation of the first units, filling of reservoir was iniciated 6 months ahead of schedule.
Therefore the construction schedule of the earth dam became critical so that the minimum reduced section of the dam in the river channel was built at this stage keeping the crest of the dam just a few meters ahead of water level.
In drastically reducing the cross section of the dam certain risks were assumed that seepage line might have emerged on the downstream slope before the full section could be built.
In such a case plans were made to place rockfill filter on the slope where seepage would emerge.
The next slide shows the construction condition as of April 1973. You can see river channel section earth fill is well advanced



Sl. 3

and the full section of the dam was rapidly built without adverse effect.
The next slide shows the project as of the same date from another angle.



Sl.4.

At present the work is progressing satisfactorily.
There remains only 13 meters of the dam to be raised until reaching the final crest elevation.
The first unit was placed in operation last month.
The project is owned by the State of Sao Paulo government-Centrais Eletricas de São-Paulo in Brazil.
General Contractor on the job is Camargo Correa and THEMAG Engenharia has done the executive design. Thank you.

Chairman Prof. V.S.Eristov
Thank you Mr. Hsu for your contribtuion. you have made. Mr. Akimov will you please

Akimov A.A. (USSR)

The availability of clayey ground with coarse filler of crushed rock,gravel,stone and boulder is characteristic of the mountainous part of the Crimea. The strength characteristic and seepage capacity of such a soil for the core and body of a high dam (69 meters) depends in the first place upon the ratio of fine (less than 2mm) and coarse (more than 2mm) fractions. The investigation ascertained that a sharp jump towards the increase of the coefficient of permeability ($> 10^{-3}$ cm/sec) and the angle of internal friction ($\geq 30^{o}$) is observed when the fine fractions content is less than 33%. Proceeding from the above-stated a soil with 40% fine fractions content was designated for the core of the dam, it secures the seepage capacity ($< 10^{-6}$ cm/sec). The strength properties in such soils depend on the clay bond, and coarse fractions (our assumption) are in a 'floating' state.
By means of rheological calculations on the basis of laboratory investigation of the soil viscosity it is ascertained for the exploitation case the biggest permissible fractions in the core of a dam- 200mm (under existing conditions).

The selection and designation of strength characteristics of soils were made by us depending on density-moisture content-stress condition. The investigation of soils were made in the laboratory with the help of Maslov-Lourier direct shear apparatus and a specially designed big shear apparatus (600 mm working ring diameter) with zone shear plane,enabling to determine at the weakest cross-section the shearing strength of soils with coarse inclusions.
The research technique of strength characteristics from density-moisture content-stress condition proceeds from the determination of boundary conditions of soil density by the standard packer apparatus of the Road Research Institute and with the correction of this density depending on the stress condition by the consolidation devices and more precise definition by recalculation or by an experiment of optimum values of humidity providing non-admission of pore water pressure during the period of the ultimate stress condition at the investigated cross-section of the dam for the existing type and mineralogical composition of the soil. Knowing the boundary conditions of density and moisture content, one can determine for them the minimum and maximum possible values of the angle of internal friction and adhesion and by them one sets the design characteristics from the stress condition and the way of work production.
We raise a problem of determining the limiting maximum value of strength characteristics under possible optimum values for a given stressed-strained state and by them we select the methods and technology of getting reliable ultimate values of the angle of internal friction and adhesion which secure the stability of a given earthwork during the periods of construction and operation.
The procedure in our opinion gives an opportunity to control strength characteristics of the soil, to reduce the volume of investigations and to rais· their quality.

Chairman Prof. V.S.Eristov
Than you Mr. Akimov. The next will be Mrs. Burenkova(USSR)

Burenkova V.V. (USSR)

Dear Chairman,dear participants of the Congress, You have just listened to a number of interesting reports concerning stress-strained state of earth and rockfill dams delivered by prof.Eristov, Rasskazov and others.
As for the stability of dams from the leakage point of view it is the most important thing to be aware of stress-strained state in antiseepage installations i.e. clay cores or diaphragm walls.
Some cracks are known to have been opened in clay cores of dams. Among them are the most dangerous ones such as transverse cracks, running in different directions,mostly from upstream to downstream direction. More than two decades ago prof.Casagrande drew attention to the fact that under certain conditions the impervious cores of dams can develop

transverse cracks and that probably the most frequent cause of the phenomena is the combination of tension and shear forces resulting from differential settlements. He warned that this complex phenomena could not be detected easily.

So, neither the advantageous methods of designing nor well graded materials could lend themselves to a simple solution and guarantee against the propagation of cracks in clay materials.

The available methods of calculation help to reveal the zones of critical stresses where cracking may develop to a predicted depth. But we are not yet in a position to predict the number or width of cracks that is why designers still have to rely largely upon precedent in seepage analysis.

At the VODGEO Institute,following the traditions established by prof.A.A.Nichiporovich we are carrying on a complex investigations for designing embankment dams. These include physico-mechanical properties and deformability of clay and rockfill materials under static and dynamic loads, stress-strained analysis using the finite element method as well as photo-elastic method and analysis of leakage through shoulders and transition zones of embankment dams with particular attention paid to antiseepage strength of clay cores. We consider transition zones to be the most important elements in earth and rockfill dams when cracks would tend to propagate in cores and diaphragm walls.

We have worked out some specifications on better grading of filter materials for dams with a clay core or a membrane.The work has been done under guideness of V.S.Istomina Dr.Sc..

Several high dams are under construction in the Soviet Union such as Nurek Dam, Viluiskaya Dam and others. Clay materials are widely used in these dams that is why we are in need to assure positive protection against piping failure through these dams. In this connection I would like to mention Thanikachalam and Sakthivadivel's paper "On the application of queueing theory to design of filter thickness". in which the authors deal with the probability analysis which considers migration of the fine particles in the filter pores. The phenomena must be a very influencing one in designing the filter width. So, the method used in the work of the Indian authors is a perspective one.

At the VODGEO Institute we also dealt with this phenomena both in the field and laboratory investigations and our proposals of the filter thickness based on the above assumptions gave us considerable economy. Thank you.

Chairman Prof. V.S.Eristov
Thank you very mush Mrs. Burenkova for your discussion. The next will be Mr.Mojevitinov

Mojevitinov A.L. (USSR)

Mon bref discours a pour but d'informer les participants du Congres de ce que la methode generale, proposee par moi et M.Chintemirov, mon jeune collegue du Kazakstan, pour le calcul de la stabilite des talus de barrages est valable, en particulier pour la resolution de tous les problemes principaux relatifs a la stabilite des barrages en terre.

Cette methode est publiee dans "Izvestia VNIIG", 1970, vol.92. La methode generale se caracterise par ce qu'elle peut etre employee pour les sols heterogenes, pour les forces actives inclinees, pour les surfaces de glissement de forme arbitraire. Cette methode se rapporte aux methodes bien connues dans lesquelles le prisme de poussee est divise en elements bidimensionnels (plans), mais contrairement aux autres telles methodes elle respecte rigoureusement, dans son entier, les conditions d'equlibre du prisme en etat limite. A cause de sa simplicite relative cette methode donne,en cas de necessite la possibilite de calculer les talus sans avoir recours aux calculatrices electroniques.

Etant donne la grande variete des schemas de calcul des barrages en terre,nous pensons qu'il faut les calculer a l'aide de cette methode ou de methodes pareilles a caractere general; en ce qui concerne les methodes de calcul statique, elles doivent des etre unifees dans tous les cas.

Les problemes classiques de la stabilite des talus aval ou amont du barrage,compte tenu des forces de capillaires et de filtration, y compris les forces dues a l'abaissement rapide du niveau dans le bief amont ou aval qui se rapportent aux schemas de calcul principaux sont resolus par notre methode generale. Ces problems peuvent devenir plus compliques en cas de presence des couches de sol tendre dans les fondations des barrages. Les forces capillaires et de filtration, comme toutes les autres forces,doivent etre determinees dans telles methodes pour chaque element du prisme de poussee separement.

Ce sont les calculs de la stabilite d'une recharge aval du barrage avec un noyau et les calculs d'un revetement du talus amont du barrage avec une masque en terre qui peuvent etre consideres comme les problemes particuliers. La resolution de tels problemes devient plus compliquee dans le cas de la consolidation inachevee du noyau et de la fondation du barrage ainsi que de la pression interstitielle exedente. Ce cas exige un calcul prealable de la filtration non permanente imputable a la consolidation du sol et au remplissage du bief. Cependant, cela n'empeche pas l'application de notre methode generale.

Le calcul du barrage aux actions seismiques peut etre reduit au calcul statique a l'action des forces d'inertie ayant une orientation defavorable, compte tenu des variations de pressions d'eau dans les biefs. Il faut dire que l'application rapide des forces seismiques est suivie par les oscillations simultanees de la pression interstitielle dans le corps et la fondation du barrage ce qui donne l'attenuation de l'effet des forces seismiques. Pour un tel calcul on peut employer aussi notre methode generale.

La diversite des schemas de calcul decrite ci-dessus ne permet pas de se borner a l'emploi d'une des vieilles methodes bien connues qui n'analysent que les surfaces de glissement cylindrique a directrice circulaire ou les forces verticales actives. Je repete que la methode de calcul doit avoir un caractere general, le critere de stabilite etant unifie pour tous les schemas de calcul. La methode de calcul de tels ouvrages importants qu'un barrage en terre doit etre exempte d'erreurs statiques caracteristiques pour les methodes approchees bien connues.

Chairman Prof. V.S.Eristov.
Thank you Mr.Mojevitinov. Now we'd like to hear Mrs.Avakian (USSR)

Avakian L.A. (USSR)

The Georgian Scientific-research Institute of Energetics and Hydrotechnical Constructions has been studying for a number of years the structural properties of the coarse waste ground as the body material for the earth-fill and rock-fill dams.

Nearly in the whole districts under study the quarry material is presented by coarse waste grounds of different origin.

In the present paper we offer the results of the above mentioned studies of the physical and mechanical properties of the coarse waste grounds, some of their quantitive characteristics for a number of constructions such as the high-head Inguri Dam, H=300 (the earth-fill version). The examined grounds were mostly presented as aggregations of a clumpy and rock debris or a boulder and pebble composition with different fillers from sandy loam to clay.

The studies were carried out both in the natural quarry materials and on their artificial mixtures.

Due to the lack of basic physical and technical characteristics of the coarse waste grounds in the existing standards and insufficiency of their research methods, we have developed special methods enabling us to carry out experiments on the grounds containing the skeletal component with the diameter up to 80mm. Special large instruments, single-axial and threeaxial were designed and made for the study of the physical and mechanical properties of these kinds of grounds.

The study of the physical and mechanical properties of the coarse waste ground was based on the external as well as internal caracteristics.

The internal characteristics are:
1. diameter of a stone,
2. amount of the skeletal component,
3. composition of the filler.

The external characteristics are:
I. density, humidity and loading under which the investigations were carried out.

In accordance to the existing standards the fractions less than 2mm were considered as alcurite, while the fractions over 2mm belonged to the coarse waste formation.

The percentage of the skeleton added to the aleurite in the artificial mixtures varied from 10 to 90.

The ground studied was mainly of the rounded type-the terrace and floor plain alluvium; more seldom there were the debris cones, debris tabus.

The bearing power was investigated for various fillers, from sandy loam to heavy loam. Experiments have demonstrated that the havier the composition of the filler, the greater the effect of the skeleton. Peaks are observed within 50-60% content of skeleton for all kinds of fillers, while for the growth of the skeleton curves became flatter; however, the curve for the ground with a heavy filler tends to go up with its content increasing over 50-60%.

The study of deformation of the coarse waste ground has shown that the more the skeleton in the ground, the less the compression modulus. A sharp drop of the modulus is observed within 60-90% content of the skeleton for all the compositions of the fillers under study. Addition of a skeletal component to the sandly loam doesn't lead to such a reduction of the compression modulus. At a normal pressure, 3.0 kgs/sq.cm, for all kinds of fillers with the sandy loam, the moduli reduced by 9-10%.

It would be interesting to present here the results of experiments on the compression of the coarse waste ground containing stones with 80mmdiameter, compressed to 50.0 kgs/sq.cm and containing the skeleton from 30 to 70% for various fillers. The experiments show clearly the regularity of the effect of the filler's composition on the compression modulus.

It should be noted that the compression degree for time, as well as coarse waste kinds of ground is affected by their preliminary consolidation by the dynamic or other kind of a consolidating load, i.e., the degree of the dense packing before the static compression, as well as the account of stones and their location in the working chamber. We have carried out a series of such experiments in the study of the quarry ground for the rock-and-earth version of the Ingury power station dam being erected now as on arch dam, H=270m. Such data have been obtained both for the rolled and angular skeletal material. However, the properties for different fillers and similar skeletons are different, though, all the curves display a common regularity. The growth of the skeletal amount is accompanied by the growth of angles, the reduction of the compression; moduli, the growth of the filtration ratio. The optimal skeletal content for all the compositions is 50-60%. However with the addition even of 30% of the skeleton, the properties characteristic for the coarse waste material are developed.

Thus the physical and mechanical properties have been investigated for different genetic varieties of the coarse waste kinds of ground. These data are now being generalized for their further classification according to the requirements of dam construction.

Eristov V.C. (USSR) , Chairman

Ladies and Gentlemen! Dear colleagues and comrades!

The time of session of our section is up and unfortunately I am to break up the meeting. I am saying "unfortunately" because not everybody who wished have had this opportunity to report and therefore not everything have been reported.

But all that have been said manifests about great investigating work which is being carried out in all countries in the field of construction of earth and rockfill dams.And this fact may be justified. The thing is that dam construction is tightly connected with the multipurpose utilization of water resources by filling great reservoirs with fresh water for the purpose of irrigation, generation of power, water supply, fish economy, transportation and construction of recreation zone for rest, going in for sports and tourism.

And I'm sure you know that the expedient utilization of fresh water resources is "Problem No.1" to-day in all countries.

And our task is to solve this problem by the most reliable and economic way for the prosperity of all mankind.

We are dealing with construction of earth and rockfill dams and it is very important that they should be reliable absolutely and most economic. Therefore it is no more chance that the problem of these dams was included to the agenda of the VIII-th International Congress on soil mechanics. All we have managed to hear today shows that many complicated problems of designing and analysis of such dams are being successfully solved as a result of the investigations. New, more improved methods of analysis and investigations are being worked out and introduced,that make us be sure of the reliability and stability of the highest dams erected of various constructing materials both at the static and dynamic loads.

Everywhere field surveys and studies of these dams behaviour in the process of their construction, at the period of filling reservoirs and commissioning are being conducted and the same time they serve as the basis for checking up and refinement of design prerequisites and methods of analysis.

At the session a number of rather interesting things have been spoken and reported some data of a great interest.

It may be stated that the work of our section may be considered as an extremely fruitfull because we have got much useful and reliable information that will certainly be used by specialists in their further work.

I must inform the participants that all the reports made on the session that have been handed over to the Congress secretariate and drawn up properly by the authors (600 words per offset sheet) and the materials of the to day's discussion will be published in volume IV of the Congress proceedings, that will be issued in some time when the Congress is over.

You are aware of the fact that according to the Congress itinerary a number of tours to various parts of our country will be organized when it is over.

I strongly recommend to the participants of our section to visit the construction site of the Nureck Hydroelectric Power Station on the Vakhsh River in the Tadjick Republik, where the highest in the world rockfill dam is under construction as I have already told you. The height will be more than 300m with clayey core, of 60 mln m^3 in volume, the capacity of the power station will be 2.7 mln. kW.

In conclusion let me thank you for active participation in the work of our section, wish you good health, success in your job and happy return home to your relatives.

Thank you.

WRITTEN CONTRIBUTIONS

MEASUREMENTS PROCESSING IN ROCKFILL DAMS

Al.Constantinescu, R.S.Comşa (Romania)

1. Measurements conducted in rockfill dams are often limited to displacement measurements because they are direct measurements of values depending on the average characteristics of the material available over a relatively comprehensive zone.

The existing equipment permits the measurement with satisfactory accuracy of both horizontal and vertical displacements at any point in the body of the dam.

2. The possible direct comparison between design data and the results obtained from measurement is of the highest interest. Such a comparison may help to specify the deformability of materials while the dam construction is in progress, wich is rather difficult to be determined by other means.

The finite element method permits simulation of the dam construction on succesive layers as well as the taking into account of certain complex characteristics, such as: nonlinear stress - strain relation, rheological properties, e.t.c.

3. To introduce in computation such properties means both to increase the number of parameters to be considered and to meet with difficulties when determining the parameters by measurements. More over, grouping in different ways the values of these parameters may result in the same value of the displacement.

4. The computation according to the assumption of a linear - deformable medium is advantageous as it requires few parameters to characterize the materials, i.e:

- E_d - deformability modulus during dam construction (succesive layers),
- ΔE - reduction of dam deformability modulus related to time - dependent deformations after the dam completion,
- n - ratio between the foundation and the dam deformability moduli,
- ν_d, ν_f - Poisson's ratios,
- γ_d, γ_f - unit weights.

5. It is suitable to use dimensionless coefficients for the processing of the results, they being appropriate for any network similar to the design one as well as for similar ratios of zone characteristics. In this case the displacements are given by:

$$u = C_u \cdot \frac{\gamma \cdot H^2}{E} \qquad (1)$$

where E is the modulus of one zone, considered as a reference zone and γ is the unit weight either of the material in the dam body (γ_d) or of the water (γ_w) according to whether the displacements are due to the dam dead load or to the hydrostatic pressure. In the case of a non-homogeneous section the unit weight and the modulus of one zone are to be considered, the problem being characterized by the ratios between the unit weights and the ratios between the moduli of the different zones (Constantinescu, Comşa and Otea, 1972, 1973).

6. The final displacements of the construction consist of:

a - displacements due to dead load during the dam construction,
b - displacements due to dead load occuring after dam construction, caused by alteration of the properties in time,
c - displacements due to hydrostatical load.

Using dimensionless coefficients determined on finite element configurations appropriate to each loading and taking into account the deformability modulus adequate for each situation, the final displacements are calculated by:

$$u = \frac{\gamma_d \cdot H^2}{E}\left\{C_u^a + \frac{\Delta E/E}{1+\Delta E/E}\cdot C_u^b + \frac{\gamma_a/\gamma_d}{1+\Delta E/E}\cdot C_u^c\right\} \qquad (2)$$

7. The distribution of the three types of dimensionless coefficients for vertical displacements in a dam cross - section is given in Fig.1, for $n = E_f / E_d = 1$

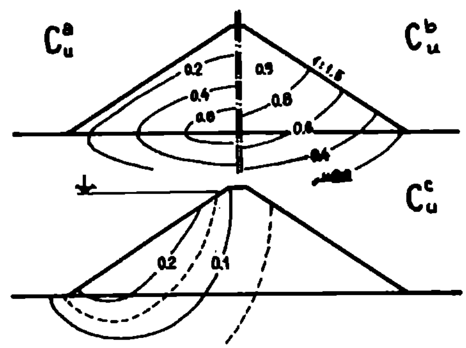


Fig. 1

8. The displacements during the construction of the dam are given by the first term of the equation (2). If the foundation is used as reference zone diagrams similar to these in Fig.2 are obtained for the values of the vertical displacement coefficients in the axis of dam cross-section. It is to be mentioned that the values of the coefficients at the foundation level do not practically depend on the ratio n . The modulus of the foundation can be determined by available measurements at the foundation level.

9. The modulus of the dam can be established by determining the ratio n from vertical displacement measurements made in the dam body. The ratio between the maximum displacement and the displacement at the foundation level depends on the ratio of the moduli, n (Fig.3).

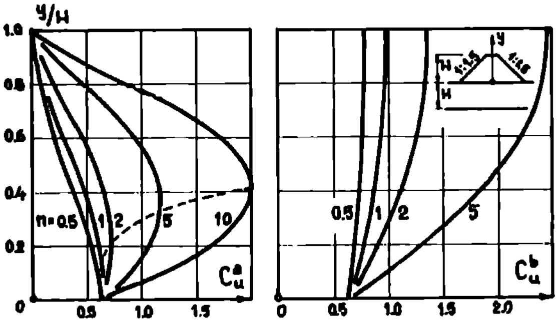


Fig.2

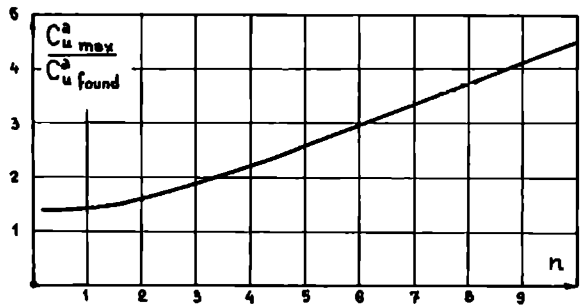


Fig.3

10. The measurements made after the construction is accomplished can be used to determine the equivalent reduction of the modulus using a finite element model charged with dead load all at once over the whole structure. The best way to determine this reduction is to use the maximum vertical displacements, recorded at the crest. The effect of the hydrostatical load on these displacements is generally negligible.

11. For evaluating the effect of the hydrostatical load the maximum displacement are the normal displacements on the impervious element (blanket, sheet, core, e.t.c.). In this case both C_u^c and C_u^b type coefficients are involved. As the values E and ΔE may be determined in advance from vertical displacements in the axis, these measurements can be used for a direct comparison between measured and computed data.

R E F E R E N C E S:

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PREDICTION OF CRACKING IN THE IMPERVIOUS CORE OF EARTH AND ROCKFILL DAMS USING THE FINITE ELEMENT METHOD (FEM). M.Dolezalova (Czechoslovakia)

Over the past years, programs for solving plane problems of continuum mechanics by FEM (on an IBM 7o4o computer using Fortran IV and on Benson-Lehner coordinate recorder for graphical output) have been developed at Hydroprojekt in Prague.

The utilization of FEM has led to further development, and to practical application of the method (1) for predicting cracking in the impervious clay core of rockfill dams.

In the designing practice of the Hydroprojekt, Prague a procedure is being implemented which is based upon determining the states of stress and strain of the dam, corresponding with the maximum values of differential settlements that may be encountered in post-construction period of the dam. For this purpose, FEM is used to examine the mentioned state in two instants of time:

1) immediately upon completing the construction under the condition that the progress of filling has been rapid so that no filtration consolidation of the core has yet been effected; the corresponding strain characteristics of clay are determined by UU-type triaxial tests;

2) in the moment when the process of filtration consolidation of the clay core will have been completed; the appropriate characteristics are determined by CID-type tests.

Solutions of multi-lift non-linear problems for these states of loading are obtained to yield integral displacement curves whose difference may be interpreted as the post-construction deformations of the dam (cf. results of "in situ" deformation measurements by means of inclinometers).

Plane problems are employed for preliminary preliminary predictions. The maximum settlement obtained from transverse problems, is projected into the longitudinal direction to determine the effect of steep rocky canyon slopes on the development of failure zones in the core. Then the values obtained by calculation are compared with experimental data.

In assessing the danger of cracking in the impervious cores of rockfill dams at Rimov (H=51.om) and at Stanovice (H=62.o m), the soil strain characteristics were carefully examined. Compressibility tests of filters and of rockfill materials were carried out in a large oedometer (D=9o cm), and clay characteristics in tensile zones were determined on beams in bending tests. The brittle behavior of clay soils under tension was respected by considering a transversally isotropic material in the tensile zones.

The solution results confirmed that some novel viewpoints must be implemented in designing dams in order to make the structure safe from cracking.

1) The degree of compaction of individual materials is to ensure homogeneous mechanical properties of the fill.

2) Design of the impervious core as well as of the filters depends on the filtration properties but also on the stress - strain properties of materials used.

3) The type and state of soil in the tensile zones are to be adapted so as to conform with the expected strain values.

Great attention is being paid to further development of FEM. Within the framework of development projects, new experimental research of the strength and strain characteristics of soils and rocks is being conducted to obtain input data for FEM (stress-paths and stress-level, volumetric strain relationships, tests under plane strain conditions, determination of Poisson ratio for tension, methods of determining the characteristics for a transversally isotropic material, etc.).

The proper programs are further developed to render the non-linear solutions more perfect (respecting dilation and contraction phenomena, plasticity and strength conditions), to solve three - dimensional problems and to automatize the computer inputs.

Confrontation of computational results obtained by the means of FEM with results on physical models and measurement "in situ" on completed structures is being prepared.

This mentioned work introduces a higher quality in the solution of the states of stress and strain of earth and rockfill dams, this being a prerequisite for more reliable predictions of the danger of cracking.

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STABILITE AU GLISSEMENT DES OUVRAGES EN TERRE - METHODES DE CALCUL. JM Dupas, JF Dies, M. Farhi, M. Londez (France).

Ce sujet très général a déjà fait l'objet de très nombreuses communications. Notre but est de montrer comment le bureau Mécasol a abordé ce problème depuis 1964, et les solutions apportées pour tenir compte des différents paramètres. On rappellera brièvement les forces qui interviennent dans l'équilibre d'une tranche de terrain (poids, forces intergranulaires et forces de l'eau en base et en bord de tranche). Les courbes de glissement étant supposées être des cercles, les calculs, basés sur la méthode des tranches, sont effectués par un programme sur ordinateur dont la principale originalité consiste en une recherche automatique du coefficient de sécurité minimal. Un autre programme permet le calcul des pressions interstitielles; il est basé sur la résolution de l'équation de Laplace par la méthode des différences finies. Il permet de faire intervenir des terrains de perméabilité et d'anisotropie différentes. Les résultats obtenus sont directement assimilables, sans intervention de l'ingénieur, dans le programme des cercles qui calcule les forces de l'eau en base et en bord des tranches. Indépendamment d'une plus grande rapidité de calcul et de la possibilité de tenir compte de nombreux paramètres, ces programmes ont permis de mettre en évidence un certain nombre de constatations qui sont discutées dans la communication.

Plusieurs méthodes permettent de déterminer la répartition de la pression interstitielle u, à un instant donné, dans les digues en terre soumises à une vidange. La méthode de Bishop est fréquemment employée, elle intéresse uniquement le talus situé sous le plan d'eau initial, sans tenir compte de l'existence de zones de perméabilités différentes dans l'ouvrage. Celles-ci interviennent directement sur la répartition et la dissipation de u .

Une autre méthode, souvent utilisée, repose sur l'application des lois générales de l'hydraulique; u est généralement calculé aux noeuds d'un réseau maillé par résolution de l'équation de Laplace à l'ordinateur, les conditions hydrauliques étant fixées par l'ingénieur : perméabilités, anisotropies, conditions aux limites.

L'application de ces méthodes suppose le sol saturé, donc des vidanges survenant assez longtemps après la mise en eau, et ne tient pas compte de l'air dissous. Dans ces conditions, les deux méthodes ne sont valables que s'il y a compatibilité entre la vitesse de vidange et le délai d'expulsion de l'eau libérée par les variations des contraintes neutres et intergranulaires.

Dans les vidanges presque instantanées (intumescences dans les canaux) ou très rapides (station de pompage) l'expulsion d'eau et d'air n'a pratiquement pas le temps de se produire. Pour approcher ces cas, les auteurs proposent une méthode de calcul de u qui consiste à analyser la relaxation du complexe formé par les grains solides, l'eau et l'air occlus et dissous.

L'ingénieur peut ainsi disposer d'un éventail de méthodes permettant de mieux tenir compte des rapports entre la vitesse de vidange et les perméabilités de l'ouvrage.

Les méthodes décrites ci-dessus ne tenant pas compte des forces intergranulaires en bord de tranche, un troisième programme a été mis au point pour étudier leur influence. Il est basé sur les considérations développées par M. Sherard dans son livre "Earth and Rock fill Dams". Malgré un certain nombre de conditions supplémentaires permettant de restreindre le domaine de variation possible du coefficient de sécurité, le problème reste hyperstatique, et le coefficient de sécurité minimal est obtenu à l'aide d'un programme d'optimisation linéaire dans le cas où l'inclinaison des forces intergranulaires est variable d'une tranche à l'autre.

Le coût d'une étude de stabilité est fonction de nombreux paramètres, en particulier de la plus ou moins grande complexité du talus, de la largeur de tranche utilisée, etc. A titre indicatif, le coût d'une exploitation se situe actuellement entre 1 000 F et 2 500 F, compte non tenu du temps nécessaire à la programmation mais qui est faible. Le nombre de cercles calculés par exploitation est souvent de l'ordre de 3 000 à 4 000. Le prix de revient d'un cercle, qui dépend évidemment du nombre de cercles calculés, est généralement de 0,40 F à 1,00 F

L'utilisation de ce programme depuis 1964 nous conduit à formuler quelques remarques.

Si l'on trace l'ensemble des cercles ayant un coefficient de sécurité au plus égal à 1,05 fois le coefficient de sécurité minimal, on constate qu'il couvre une zone qui doit être effectivement la plus critique. Les surfaces de rupture, circulaires ou non, situées à l'intérieur de cette zone, devraient conduire aux coefficients de sécurité minimaux.

Il nous a paru intéressant de comparer la méthode ci-dessus à celle se basant sur les courbes de rupture polygonales. Un des avantages souvent avancé en faveur de cette dernière méthode est la possibilité de faire passer la ligne polygonale dans une couche de faible épaisseur, alors qu'un cercle ne pourrait s'y inscrire. Dans le cas où l'on travaille en rupture circulaire, on peut remédier à cet état de fait en introduisant dans les calculs une couche de plus forte épaisseur que la couche réelle. Cet artifice nous paraît à recommander dans le cas de talus naturels pour lesquels la stratigraphie est moins bien définie que dans le cas d'un remblai réalisé par l'homme. Nous n'avons pas effectué d'étude comparative systématique des résultats obtenus par la méthode des glissements circulaires et par celle des lignes polygonales A titre indicatif, pour un barrage zoné, le coefficient de sécurité du talus aval en régime permanent est en glissement circulaire (méthode standard des tranches) FS = 1,57 alors qu'en lignes polygonales il est FS = 1,72. Le profil semblait favorable à l'utilisation d'une ligne polygonale (couches assez minces en fondation de caractéristiques plus faibles) et pourtant la méthode des glissements circulaires a conduit à un coefficient de sécurité assez nettement inférieur sans majoration de l'épaisseur des couches faibles. Il ne faut évidemment pas généraliser ce résultat.

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COMPACTAGE DES SOLS COHERENTS PAR LA CHARGE CYCLIQUE.

A.N. Ermolaeva, B.F. Reltov, U.R.S.S.

A present les valeurs optimales de la compacite et de l'humidite des dispositifs d'etancheite (noyau, masques) des barrages en terre mis en place par couches sont determinees prealablement en laboratoire a l'aide de la methode de Proctor. Dans un nombre de cas les valeurs prealables de la compacite et de l'humidite different considerablement des valeurs reelles obtenues lors du compactage du sol par les rouleaux. Cela s'explique par le fait que lors du roulage la charge de compactage a un caractere cyclique, et outre cela la dilatation laterale du sol est possible. Les parametres de la charge cyclique dependent du type de l'engin compacteur employe et du regime de roulage. Dans l'appareil de Proctor le compactage du sol est assure par les chocs d'un certain poids, le sol etant enferme dans une enveloppe rigide qui elimine la possibilite de la dilatation laterale. Les recherches de laboratoire portaient sur la mise en evidence de l'influence d'un des parametres les plus importants de la charge cyclique, c'est-a-dire de sa valeur maximale dans les conditions ou la dilatation laterale est possible, sur la compactibilite du sol. On a essaye la terre argileuse loeussique en etat perturbe (sans remaniement) mais ayant quelques mottes et agregats de structure naturelle. L'humidite initiale du sol compacte variait dans les limites de 17% a 30%. Les valeurs maximales des contraintes dues a la charge cyclique atteignaient 1.5; 3.0; 5.0 et 7.0kg/cm^2. La variation de la vitesse du chargement avait un caractere lineaire et etait egale dans les essais a 1 kg/cm^2 en seconde. On realisait le dechargement immediat apres le chargement, les intervalles entre les cycles etant egales a 15 min. Pour chaque valeur de l'humidite on effectuait 10 cycles charge-decharge. La fig.1 represente les courbes de variation de la compacite du sol en fonction de l'humidite pour differentes charges de compactage. Les relations donnees montrent que la compacite du sol croisse avec l'humidite jusqu'a une valeur maximale determinee qui depend de la charge. Lors de l'augmentation de la charge de 1,5 jusqu'a 7.0 kg/cm^2 on a une augmentation de la compacite optimale (poids volumique du squelette) de 1.44 a 1.64 g/cm^3, l'humidite optimale diminuant de 29 a 21%.

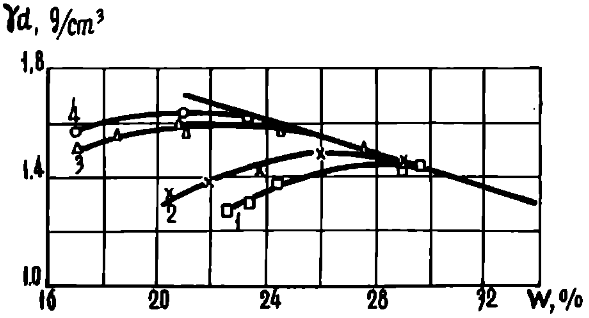

Fig. 1 Variation de la compacite en fonction de l'humidite pour differentes valeurs de la charge de compactage
1. charge de compactage egale a 1,5kg/cm^2;
2. dite, a 3 kg/cm^2; 3. dite, a 5 kg/cm^2;
4. dite, a 7 kg/cm^2

Les recherches effectuees ont demontre qu'a l'humidite inferieure ou egale a celle optimale pour la charge cyclique maximale donnee, le compactage du sol n'est pas suivi d'une dilatation laterale considerable. Ceci indique que dans ces conditions la resistance du sol ne diminue pas. On a constate que l'augmentation de l'humidite seulement a 1.0-3.5% par rapport a celle optimale (en fonction de la charge de compactage) mene a la perte de la capacite portante du sol qui se traduit par son expulsion de dessous la plaque. La tangente aux courbes de variation de la compacite en fonction de l'humidite (fig.1) correspond aux valeurs limites de l'humidite audessus de laquelle la perte de la capacite portante a lieu. Aux valeurs optimales de l'humidite et de la compacite du sol sous diverses charges, le coefficients de saturation d'eau est egal a 0.85-0.89, la perte de la capacite portante ayant lieu pour le coefficient egal a 0,92-0,96. La fig.2 donne les courbes de variation des valeurs optimales de la compacite et de l'humidite en fonction de la charge de compactage. Comme on voit ces valeurs dependent essentiellement de la charge de compactage.

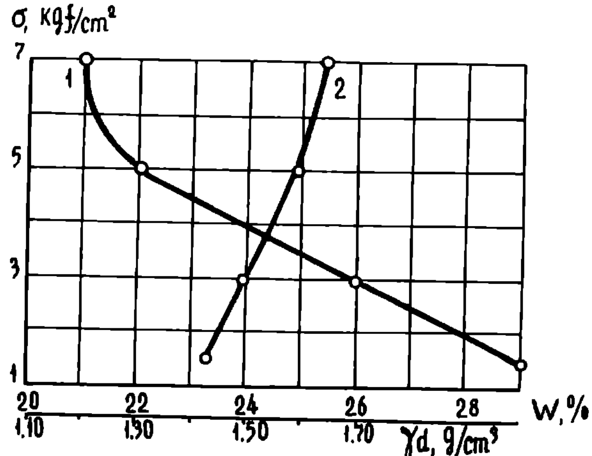

Fig. 2 Variation des valeurs optimales de la compacite et de l'humidite du sol en fonction de la charge de compactage
1. variation de l'humidite en fonction de la charge; 2. variation de la densite optimale en fonction de la charge.

Les investigations accomplies ont montre aussi que les conditions optimales n'ont lieu que pour une telle combinaison de la charge et de l'humidite qui assure la structure monolithe du sol compacte (sans macropores visibles et agregats) ayant une resistance considerable. Plus petite est la charge de compactage, plus grande est la valeur de l'humidite du sol a compacter qui est necessaire pour assurer la structure monolithe. On peut obtenir aussi la structure monolithe pour les valeurs de l'humidite plus grandes que celles optimales (pour une charge donnee), mais dans ce cas une perte de la capacite portante du sol aura lieu. Les essais de laboratoire relatifs au compaction du sol sous l'action de la charge cyclique (a parametres donnes) dans les conditions de la dilatation laterale possible donnent la possibilite de faire un choix prealable du type de l'engin compacteur en conformite avec les conditions reelles de construction des ouvrages en terre, ou de determiner les valeurs optimales de la compacite et de l'humidite du sol pour le type de l'engin compacteur choisi et le regime de compactage (c'est-a-dire pour les parametres de la charge cyclique donnes). La methode de Proctor ne permet pas de representer en similitude les conditions reelles du fonctionnement des engins compacteurs.

PROCÉDÉ DE CONTRÔLE DES DIGUES EN CENDRE.

K. Fiedler /Pologne/

Pour déposer d énormes quantités de cendre provenant des centrales thermiques à charbon on utilise souvent le transport hydraulique. Les cendres sont ainsi déposées dans des bassins de décantation dont la superficie dépasse des dizaines d hectares. Les bassins sont créés par des digues en terre, fréquemment d une longueur de quelques kilomètres. Après quelques années d exploitation ces digues sont surélevées, en général par remblai en cendre. Tenant compte de la connaissance médiocre des prioriétés des cendres et du manque d expérience dans leur utilisation comme matériau de construction, on essaye divers procédés de contrôle de la qualité du remblai en cendre.

Un pénétro-scissomètre à percussion-rotation /fig. 1/ utilisé en Pologne /Wiłun 1967/ a été appliqué pour controler la qualité des remblais des digues en cendre d un bassin de décantation d une centrale thermique. La pénétration de la sonde est assurée par la chute d un poids avec une energie égale à 55 Nm. Le nombre de coups par chaque dizaine de centimètres d enfoncement de la sonde donne une image de la résistance c.-à-d. du degré de compactage du sol ou de la cendre. En mesurant à l aide d un dynamometre le moment des forces nécessaire pour tourner le croissillon du scissomètre on obtient la résistance au cisaillement globale.

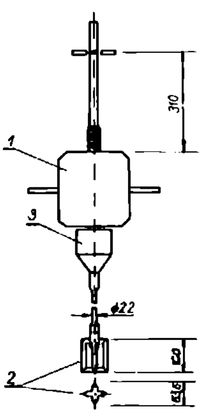

Fig. 1. Pénétro-scissomètre /type ITB-ZW/. 1-croissillon de cisaillement, 2- poids, 3-chassis du dynamomètre

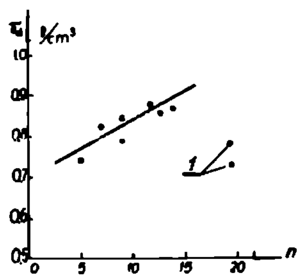

Fig. 2. Relation entre le nombre de choc par 10 cm d enfoncement de la sonde /n/ et la densité apparente sèche de la cendre /γ_d/. 1-influence d autocimentation

Il faut souligner que des phénomènes d autocimentation peuvent améliorer les prioprietés mécaniques de la cendre dans le temps. Un sondage de contrôle doit donc succéder immédiatement aux travaux de construction. Dans le cas contraire les résultats du contrôle obtenus peuvent signaler une qualité exagerée de ces travaux. L influence de l autocimentation des cendres étudiées est visible sur la fig. 2, qui représente la relation entre le nombre de chocs par 10 cm de pénétration de la sonde et la densité apparente sèche de la cendre.

Les résultats des sondages dans une zone faible du remblai en cendre étudié sont visibles sur la fig. 3a. On y voit, sur une profondeur de 2 à 4 m, des couches insuffisamment compactées. Selon une enquête supplémentaire cette zone a été construite et compactée dans des conditions atmosphériques défavorables. Une mise en eau d essai a provoqué dans cette zone des tassements locaux d environ 40 cm. Ce phénomène vérifie donc entièrement les résultats du sondage.

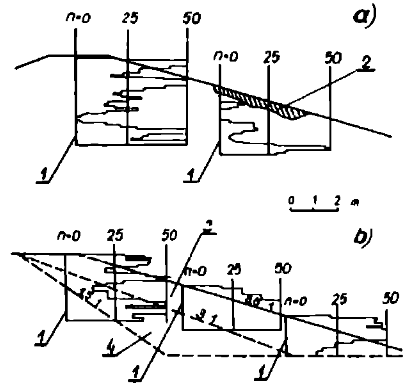

Fig. 3. Zone faible d un remblai en cendre. a/ avant, b/ apres des travaux de renforcement. n-nombre de chocs par 10 cm d enfoncement, 1-sondages, 2- tassement apres la mise en eau d essai, 3-remblai supplémentaire en cendre, 4-remblai en cendre rechargé.

Après une restauration et un renforcement du remblai en cendre /fig. 3b/ on ne note pas de perturbations dans l exploitation des digues du bassin de décantation.

CONCLUSIONS. Les essais d application d un pénétro-scissomètre à percussion-rotation pour le contrôle de la qualité des remblais des digues en cendre ont montré l utilité de cet appareil. Le sondage permet l estimation de l état du compactage de la cendre et de sa résistance au cisaillement. On peut réveler ainsi des zones faibles du remblai en cendre.

L application du pénétro-scissomètre peut donc être constaté comme un procédé assez simple de contrôle des digues en cendre, mais sous condition que le sondage succède immédiatement aux travaux de terassement.

REMERCIEMENTS. L auteur remercie MM J. Wemcewicz, F. Szczypiorski et W. Więckowski qui ont participé aux travaux de sondage in situ.

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FIELD INVESTIGATIONS OF EARTH AND PORE WATER PRESSURES IN PATCHKAMAR DAM.

I.V.Fjodorov, P.V.Dergatchov, V.V.Alipov, USSR.

The Patchkamar hydraulic structures on the Guzar-Darja river were constructed under complicated geological and hydrogeological conditions in a mountain region. The seismic degree is 9. The hydraulic structures consist of the 70m high dam, the outlet works situated at the left abutment of the dam and the emergency chute spillway. The cross section of the dam is shown in fig.1. The purpose of the dam is to form the seasonal storage with capacity 280 mill m^3 for irrigation and water supply.

The Vodgeo Institute carried out the complex investigations "in situ" from the construction period /1965-1967/ till 1971. Their programme included the determination of settlements and displacements of the dam; the control of seepage through the dam, beneath a dam base, both sides of the dam as well as the measurements of pore water pressures within the core and earth pressures within the core and on the surface of the outlet works.

The electrotensiometric transducer-type apparatus was designed for the earth and pore pressures to measure and to record as well as the technique of cell placing and measurement conducting was developed. On the whole there were the earth pressure cells-55 and the pore pressure ones- 16 to place into the dam /Fig.1/.

The earth pressures within the core were measured on three levels in the middle channel part of the dam. When an embankment above the instrument level was of h=15-20m distribution of the vertical earth pressures was practically uniform. As the embankment height was increasing up to 40m the distribution line got convex, the average pressure over the range of core width amounted to a few less γh/ γ -the bulk density of core material/ During the operation period the pressure redistributions can be observed as a result of storage water level alternations, the average pressure magnitudes decreasing further in comparison with γh. So the pressures on the first and second levels have decreased to 0.7 γh on the average, but the third level- to 0.8 γh, it seems to be due to arching effect between the core and the shells. On the second and third levels the horizontal pressures were measured together with vertical ones at three points-at the central axis and at core edges. During the construction period ratios between horizontal and vertical pressures / ξ / were 0.42-0.43 with h up to 7-10m at the given points whereas with h=20-40m ξ =0.5-0.53 at the core edges and ξ =0.42-0.49 at the axis. During the operation period ξ is 0.4-0.54 on the second level and 0.12-0.38 on the third level.

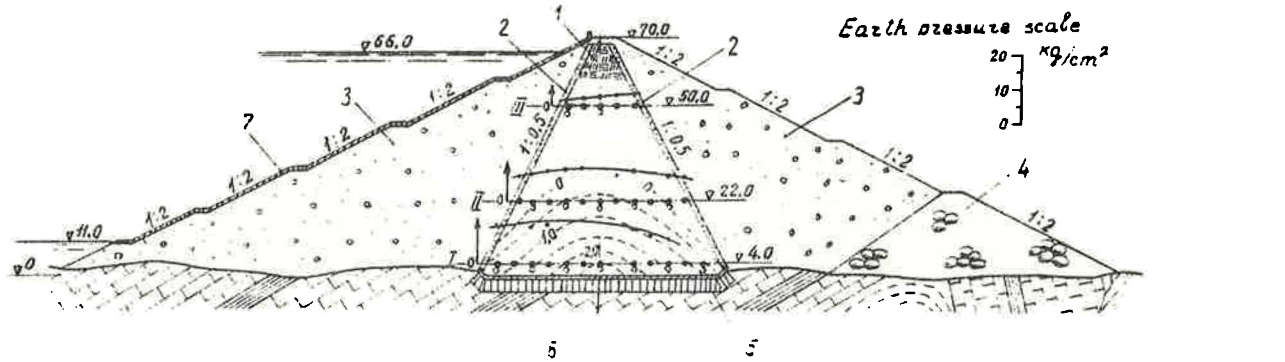
When measured, the significant concentration of the earth pressure was registrated on the vertical facing at the outlet works. The average pressure value over the range of the outlet width was above γh in the construction period. During the operation period decreasing of the pressure have been observed in comparison with γh that may the the result of the arching effect between the core and the shells as well as the bank.

The pore pressures were measured on the same levels of the same cross section as with the earth pressures. The rise of the pore pressures was fixed not long after the cell placing at h=15-20m. The pore pressure changes over the range of the core section and at all times were regular.

The maximum pore pressure were developed at the bed in the middle core part. When the construction works were under completion its value amounted to 2.6 kg/cm^2 or 0.2-0.25 γh. The maximum pore pressure on the 20m above the base was 0.6 kg/cm^2 or 0.1-0.13 γh. The excess pore pressure was dissipated more intensively within the period of the 5-6 months when the dam construction had been over. The period of the complete dissipation of the excess pore pressure lasted almost a year.

The experience of organization and the results of the field investigations at the Patchkamar dam can be used in designing of similar structures both for more accurate calculations and improvements of placing and designing the control apparatus.

FIG.1. The cross section of the Patchkamar dam 1-a loesslike-loam core; 2-transition zones; 3-sandy-gravel shells; 4-a rockfill toe; 5-an area of cement grouting; 6-a low pervious layer (clay); 7-a concrete facing; ●-earth pressure cells; O -pore pressure cells.



—vertical earth pressure curves; - - - lines of equal pore pressure (kg/cm^2) at the end of construction period.

CONDITIONS DE FORMATION DES FISSURES DANS LES BARRAGES EN TERRE. P. HABIB, M.P. LUONG, Q.S. NGUYEN (FRANCE)

Les barrages en terre, après construction et mise en eau sont le siège des déformations différées de tassement accompagnées parfois de fissurations visibles. La présente étude montre qu'avec des hypothèses relativement simples, on peut représenter avec une assez bonne approximation des effets constatés sur des ouvrages réels et prévoir la position et l'amplitude des désordres en surface. Elle permet aussi de suivre la progression des désordres à l'intérieur du barrage dans des zones où l'observation est pratiquement impossible.

Le sol, à l'instant initial, est caractérisé par des coefficients d'élasticité E et ν . Sous l'action du poids propre et lorsque l'eau d'infiltration en provenance de la retenue humidifie le sol, on constate une diminution de volume, qu'on peut assimiler à une augmentation de la compressibilité. On définit ce nouvel état par E' et ν', et cet effet est supposé instantané.

Le coefficient de Poisson semble peu affecté par la teneur en eau ($\nu = \nu'$) ; les variations de densité sont évidemment faibles, la résistance au cisaillement est donc pratiquement constante. Le critère de rupture reste alors celui de l'état initial défini par une relation de Coulomb $t = c + n \, tg\phi$.

On détermine par le calcul les déplacements et les contraintes en tous points sous l'effet du poids propre et des forces extérieures appliquées à l'ouvrage en envisageant des zones humides de différente étendue, fonction du temps ; on compare ensuite les contraintes au critère.

Pour l'exploitation du calcul, on choisit une courbe intrinsèque et on évalue le danger de rupture R_F en chaque point par le rapport du rayon R du cercle de Mohr à la distance D du centre de ce cercle à la droite de Coulomb. (Figure 1).

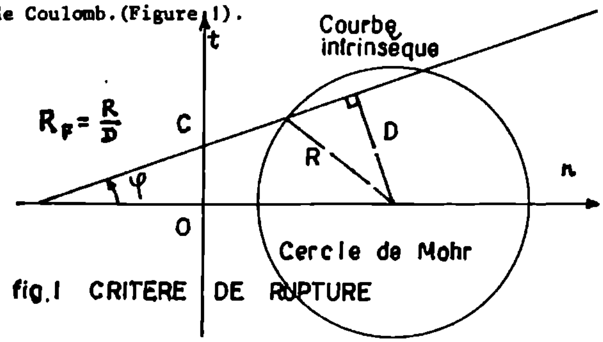


fig. 1 CRITERE DE RUPTURE

Pour $R_F < 1$, l'état de contrainte est élastique.

$R_F \geq 1$, le critère de la courbe intrinsèque est violé ; le risque est d'autant plus grand que R_F est plus élevé.

Mais la rupture n'est pas la fissuration : il peut simplement se produire des déformations plastiques. Si la contrainte moyenne est forte, les déformations sont du type ductile. Si elle est faible, des lignes de glissements peuvent apparaître tout en maintenant en contact les lèvres de la cicatrice ; par contre, l'apparition d'une contrainte principale de traction laisse craindre l'ouverture d'une fissure. Ceci montre la nécessité de distinguer dans l'appréciation de la rupture les zones de fissuration et les zones de plastification.

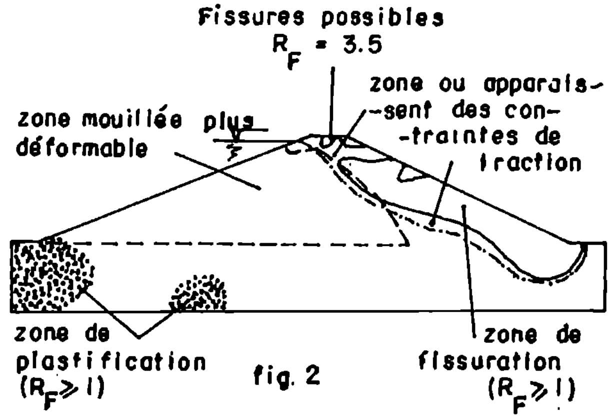


fig. 2

SCHEMA DE CALCUL ET D'EXPLOITATION DES RESULTATS

La figure 2 schématise un barrage homogène en terre à base imperméable soumis à la pression d'eau de la retenue. Sous l'action des tassements de la zone d'invasion plus déformable, on peut voir se former des zones de plastification ($R_F \geq 1$, critère de rupture violé en compression) et des zones de fissuration ($R_F \geq 1$, critère de rupture violé en traction). Les fissures peuvent se former là où la valeur de R_F est grande.

Les résultats obtenus ont clairement mis en évidence les influences dues aux variations de l'étendue de la zone humidifiée, des caractéristiques mécaniques (élasticité et rupture) et ils donnent des tassements tout à fait concordants avec les observations.

Pour serrer de plus près la réalité, des améliorations sont possibles en tenant compte par exemple de la construction par couches pesantes successives.

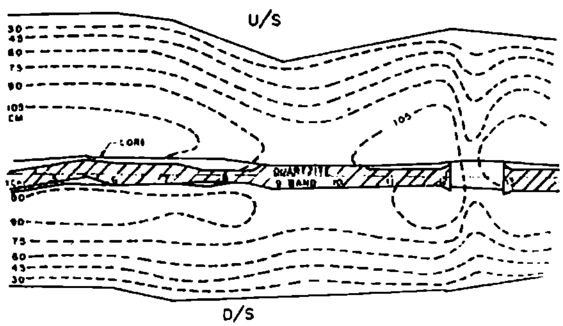
Quoi qu'il en soit, les résultats obtenus sont en accord avec les constatations des désordres réels observables. Mais ils signalent en plus des possibilités de désordres dans des zones mal observées (le talus aval) ou inobservables (l'intérieur du barrage) où des organes comme des filtres ou des masques d'étanchéité peuvent se trouver.

DESIGN, ANALYSIS AND END OF CONSTRUCTION SETTLEMENT STUDIES OF A ZONED EARTH DAM ON NONUNIFORM FOUNDATION. R. K. Katti and P.N. Sundaram (India)

The problem regarding possible development of longitudinal and transverse cracks was faced in case of a 600m long and 30 m height dam planned and constructed across river Garvi in Central India. This was due to the presence of a nearly vertical quartzite band of width 13-30m wide running parallel to the dam axis except between Ch.12 and Ch.13 and is flanked by alluvium underlain by Kaolinitic clay of thickness 10-20m having compressibility index varying between 0.05 and 0.16. The material used for the construction of zoned dam, was locally available silty or clayey sand, SM-SC or CL materials and poorly graded GP-GC material.

Three sources of settlement were envisaged in the dam: (a) compression of the dam structure, (b) volume change on initial filling of the reservoir and (c) foundation settlements. Computation based on considering material characteristics and period of construction including monsoon break etc., in general showed that the settlement due to condition mentioned under (c) would be critical due to the presence of quartzite band.

Stresses in the foundation were computed using Pol'shin solution (Florine, 1959). Special computer programmes were developed to evaluate the total settlement using finite difference technique incorporating equivalent step loading, monsoon break and variation in the compressibility of the foundation material. Settlement contours are shown in Figure below:



FOUNDATION SETTLEMENT CONTOURS

Evaluated differential settlement values viewed in the light of stress deformation characteristics of material used in the dam indicated possibility of development of longitudinal and transverse cracks in certain parts of the dam, which could pass through core, shell or both. Static stability, considering tension cracks was checked for critical sections.

The design alterations and protective measures considered consisted of (i) widening the core width with more plastic material but of low compressibility, compacted to slightly on wet side of the optimum, (ii) provision of thick graded filters on either side of the core and (iii) use of GP-GC material in certain portion of the shell and cover it up with sand.

End of construction settlements observed in certain instrumented portion of the dam are in close agreement with the predicted values, as shown in Table below:

		Ch.5.50	Ch.12.50
1. Construction period		6	24
2. End of construction settlement in cms	predicted	12	21
	observed	10.50	15.50
3. Total embankment settlement in cms	predicted	105	84
4. End of construction pore pressures (per-cent of total soil head)		12.6	9.7

Until recently (1973), the dam is performing satisfactorily and there is no undue increase in the seepage quantity.

The authors acknowledge the Director, IIT Bombay for his keen interest.

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ON AN EXPERIMENTAL STUDY AND NUMERICAL ANALYSES ON STRESS AND DEFORMATION OF FILL-TYPE DAM. T.Kawamoto, T.Niwa and T.Miyaguchi (Japan)

In order to research the nonlinear deformation behavior of fill-dams, the results obtained by the small-scale experimental approach were compared with those computed from the finite element analysis.

The dam model with a cross-section of right-angles triangle (2 m in height, 4 m in base length and 1 m in width) composed of Toyoura standard sand is filled in a test box considering that the hydraulic pressure is applied to the upperstream face of center core part. The dam model was loaded horizontally through 200 pieces (20 rows x10 columns) of loading jacks. The displacement of vertical loading face and the stresses in dam model were measured for various stages of loading and for several densities of sand of model.

The non-linear stress analysis was carried out by the incremental procedure using the hyperbolic approximation of the stress-strain relations based on the triaxial tests for the material of model. At the same time, the elasto-plastic analysis was performed under the assumption of Drucker-Prager's yield criteria. The results of both analyses presented the similar tendencies and the load at failure of dam model became to be 1.8 times hydraulic pressure for full reservior (described by 1.8H) in the former case and 1.6H in the latter case.

The deformations of dam body, however, were smaller wholly for the experiments than for the numerical analyses. The calculated displacements of the crest of dam model became to be about three times the experimental ones. It was recognized the differences of incremental rate of horizontal displacement between the experiments and the calculation. Such differences seem to be resulted from that the stress pathes in ordinary triaxial compression test can not represent the variations of stress in the dam model subjected to the load below 0.7H. The state of stress in the vicinity of the loading vertical face is to be $\sigma_1 > \sigma_3$, where σ_1 iis the maximum principal stress, a nearly vertical component and σ_3 is the minimum principal stress, a nearly horizontal component, for only the body force of embankment and then σ_3 increases gradually whereas σ_1 keeps to be almost constant with the increases of hydraulic pressure. Furthermore, after passing through the states of stress of $\sigma_1 = \sigma_3$ (at the loading stage of about 0.7H) , σ_1 becomes to increase gradually with the hydraulic pressure. On the other hand, the stress-strain relations in triaxial compression test are obtained for the increments of σ_1 from the state of $\sigma_1 = \sigma_3$.

It is clear from the results mentioned above that the numerical analyses should require to investigate the stress-strain relation of the material under consideration of complecated patterns of stress paths occured practically in models or prototypes.

For instance, let's group the stress changes into the four conditions like the followings as regards with the rupture envelope of materials (Fig.1);

Condition A; is fixed and increases.
Condition B; increases and is fixed.
Condition C; decreases and is fixed.
Condition D; is fixed and decreases.

Fig.2 shows the stress-strain relations of Toyoura standard sand corresponding to the stress conditions of A, B, C and D, respectively. It seems that the stress change from A to B roughly corresponds to the stress changes near the vertical loading surface on the experimental model or behind the center core of a rockfill dam. It is noted that the tangent to the stress-strain curve in the condition A in spite of the decreasing of the the principal stress defference. If such a deformation property dependent on a stress path is introduced into the finite element analysis, the deformation behavior of the experimental models of the prototype dams may be explained.

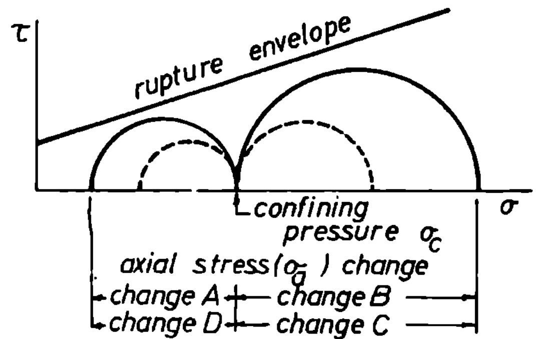

Fig. 1

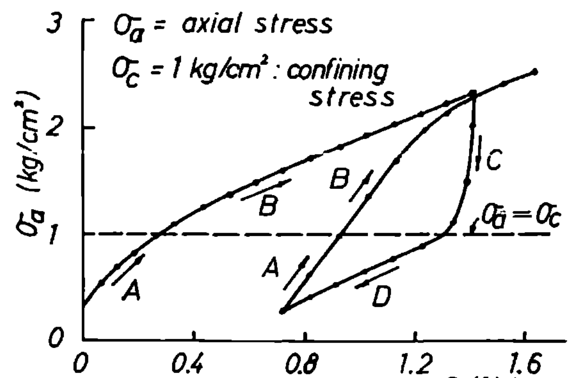

Fig. 2

Strains of the Up-river Retaining Prism of a Dam under Filling. M.A.Kolosov (USSR).

In the report there is a method of calculation of vertical shrinkage strains of the up-river retaining prism of a dam and accompanying them horisontal displacements of the crest, arising during the procedure of the fill of the reservoir. The working out of the method has followed the experiments to investigate shrinkage characteristics of large fragmental material. It was done in laboratory of construction of Krasnoyarsk Water Power Station. The experiments were done in a steel cylinder d=205 millimeters h=237 millimeters under gravitation load.

According to the results of the experiments linear dependence of shrinkage strains on material density, taken into consideration for calculation was established.

$$e = k(\varepsilon - \varepsilon_0) \quad \text{........ (1)}$$

in which e - relative shrinkage strain;
ε - test material density, expressed through porosity factor;
ε_0 - material density, in the presence of which shrinkage strains are equal to 0,
k - test ratio.

Density change according to the depth of an embankment was accepted in accordance with the nature of compression curve of material and is expressed with exponential formula

$$\varepsilon = \varepsilon_1 - \alpha p^\beta \quad \text{......... (2)}$$

in which ε_1 - material density on the surface of embankment after compression;
p - load;
α, β - test ratioes, obtained under the working of compression curves.

Load influence on the magnitude of shrinkage was noted during the experiments, of two from six mixtures of test materials, during the increase of load the increase and decrease of shrinkage have taken place. The influence of load was counted up with the exponential formula as additional change of shrinkage.

$$\Delta e = \pm a p^n \quad \text{.........(3)}$$

in which p - load;
a, n - test ratioes

Without the account of the load influence on shrinkage strains the magnitude of shrinkage of up-river prism under the level of submergence y_i is determined

$$\Delta(y_i) = k y_i(\varepsilon - \varepsilon_0) + \frac{k\alpha\gamma^\beta H^{\beta+1}}{\beta+1}\left[(1-\bar{y}_i)^{\beta+1} - 1\right] \quad \text{...(4)}$$

in which: H - dam high;
$y_i = \frac{y_i}{H}$ - degree of its submergence;
γ - volume weight of prism material.

In the report there are formulas of shrinkage of up-river prism deduced with calculation of load influence under both partial and complete submergence, as well as formulas of shrinkage with the calculation of density change of prism material owing to strains of creep, if filling of the reservoir with water takes place not after filling a dam immediately, but in some years time.

Due to irregular shrinkage strains in prism some horisontal displacements of dam crest take place: transverse displacements - to upper water, longitudinal ones - from borders to axis of canyon. The experiments in a trough with transparent wallside and observation of strains embankments in the zone of submergence of Krasnoyarsk Water Power Station have helped to find out the procedure of horisontal strains. It was established, that horisontal displacements take place in two cases:

1) under submergence of embankment with a slope, that leads to displacement of slope edge;
2) under submergence of embankment laid on the inclined foundation - bed, that leads to cracking on its surface.

The first case corresponds to the crest displacement to upper water, the second one - to longitudinal displacements of crest.

Taking into account that horisontal strains are the result of turning of prisma dry body owing to irregular strains of its submerged part we found the formulas of the magnitude of displacements to the upper water and longitudinal displacements of crest from borders to the axis of canyon.

Compared calculations made for embankment H=50 m from tested compositions, and calculations for displacement Infernillo dam give rather similar results to natural data both in quantitative and qualitative aspects.

ANALYSIS OF THE EFFECT OF THE MAIN FEATURES OF DEFORMATION BEHAVIOUR OF SOILS ON THE STRESS-STRAIN CONDITIONS OF DAMS OF LOCAL MATERIALS. A.L.Kryshanovskij, A.S.Chevikin, E.G.Pikus, M.M.Pashchenko.

As differed from the model of linearly deformable medium the most frequently used in the engineering calculations, finely-divided and coarse-skeleton soils exhibit a number of features of their deformation behaviour: the shape and volumetric deformations are non-linear with the linear (I) and square (II) invariants of the stress tensor. Furthermore, there is substantial influence of the Nadai-Lode parameter of the type on three dimensional state ( ) the loading path and the rotation of the principal stress axes in the course of deformation /1,2,3/.

The loss of the local stability and crack formation may be predicted in those zone of a dam of local materials, where the ultimate equilibrium state is established. Therefore, the most critical points in the calculation of dams by the methods of the linear soil mechanics are of the least reliability. Where the main features of the deformation behaviour of the dam, soils are but partially reflected in the calculation, the analysis of the effect of the factors, which have not been taken into account, is required. Yet there are no sufficient full-scale observation data (especially as to the crack formation for the purposes of such an analysis.

For that reason it is necessary to develop the methods of calculation which would permit to more accurately reflect the deformation properties of the dam soils.

The studies show that the role of the parameter and dilatancy phenomena is very important in the deformation regularities. Thus, under the pure shear deformation conditions the invariants I and II can be increased by 5-10 times (4) due to the manifestation of the dilatancy properties.

The main principles of the calculation are the following. The mechanical properties of the material under the quasi-equilibrium deformation have been determined using the apparatus ensuring the application of three independently variable stresses under the pure deviatoric loading conditions. The coaxiality of the stress and strain conditions during the test was ensured by the structure of the apparatus which is reflected in the calculation. In addition, the similarity between the stress and strain conditions was assumed in the calculation. The resulting relationships have been approximated. The moduli of deformation (Lame parameters) depending on the stress-strain conditions have been determined as reported by I.A.Birger /5/ as extended to take into account the influence of and the volumetric deformation under the action of a stress deviator. The plain strain case of the dam having an incompressiblo base has been considered. The calculation has been made by the finite difference method using the BESM-4 computer. The boundary conditions have been predermined as earlier reported /3/.

The calculation results will be considered taking as an example the simplest case of the dam built of one and the same material. In the first example the deformation properties are taken into account including the dilatancy phenomena; in the second example the dilatancy phenomena are not considered. The solution has been obtained for the Nurek dam (H=304 m).

The calculation results show that the effect of the dilatancy attains 30%.

This results in the conclusion that the dilatancy must be taken into account in the engineering calculations.

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OBSERVATION OF EARTH DAMS IN PORTUGAL.

E. Maranha das Neves and F. Guedes de Melo
(Portugal)

Having begun more than ten years ago to install observation equipment in earth dams the Laboratório Nacional de Engenharia Civil has collected data, whose interpretation seems of interest to make known.

The structures observed can be divided in two main types: dams made up of weathered schists with a clay core (Mira and Monte-da-Rocha dams); and clay fills in composite dams of both the earth-buttress (Caia and Roxo dams) and the earth-multiple arch types (Odivelas dam). The latter, in addition to the usual interest of observation results, supplies data on the behaviour of earth fills in contact with concrete blocks. In Mira and Monte-da-Rocha dams the shells are made up of materials with a wide range of grain sizes and a high percentage of coarse aggregate (50 to 150 mm). These fills were compacted by vibration rollers in layers about 1.0m thick. The study of the behaviour of these fills rendered field measurements indispensable, mainly of pore pressure and soil deformability.

The observation equipment used included among others: electric and hydraulic piezometers, internal movement devices (U.S.B.R. and slope indicator types), and surface settlement points. The good performance as regards both response and durability is especially noteworthy. Even when installed in zones of the earth fill where pore pressures due to the impounded water easily exceed 4 kgf/cm^2, they remain in good operating conditions seven years after their installation. It is also important to notice the extremely strong trend to the installation of negative pore pressures duringconstruction, which subsist in considerable zones of the earth fill even after years of water impounding in the reservoir.

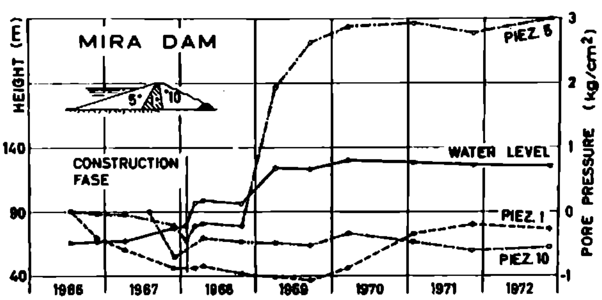

FIG. 1

Typical behaviour of 3 electric piezometers is presented in figure 1. They were also choosed in such a way that they are from the tree principal zones of the dam: core, upstream and downstream shells.

The data on the characterization of the deformability in earth fills are mainly drawn from the observation of Mira dam (90 m high, 4×10^6 m^3 of earth) and Monte-da-Rocha dam (52 m high, 2×10^6 m^3 of earth). These are earth dams with a core, in which both the materials and the construction (compaction) methods are different from those used in the

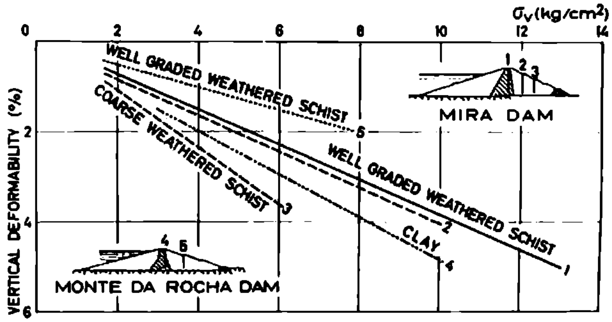

FIG. 2

shells. Figure 2 presents the observed internal vertical displacements measured with the instaled U.S.B.R. assemblies type. The results are shown as the modulus of vertical deformation in function of the vertical pressure, considering this one as the weight of the overburden soil.

In both dams two zones are observed: the core and the downstream shell. As can be seen in the figure, there is a wide range of deformability depending on the soil type and on the relative stiffness of neighbour zones.

As a normal behaviour the coarse material in the shell of Mira dam (curves 2 and 3) is more deformable than the weathered schist of the core (curve 1); but a higher differenciation might be expected between curves 1 and 2. In fact, given the large deformability of the coarse schists, the stresses on the shell zone near the core are greater than those which would occur in a homogeneous fill. So, if the correct stresses were used in the plot, curve 2 would move to the vicinity of curve 3.

In the Monte-da-Rocha dam it happens the inverse situation: a clay core (curve 4) have higher deformability than the schitous shell (curve 5).

A global analysis of the deformabilities points out the good performance of the well graded weathered schist employed either in the core or in the shell zones. The referred analysis is also of the utmost interest, notably as — owing to the differences displayed — deformabilities give rise to a considerably redistribution of stresses between the core and the shell.

In Roxo dam, the important settlements observed in the earth-concrete contact, shows that construction details and seepage conditions in that zone must be carefuly studied.

Finally the results of the observation of surface settlement points and the displacements measured are related with certain solicitations on the completed structure. The most important facts are a sort of a little global circular motion in Mira dam resulting from water impunding in the reservoir and the upwards movement measured near the concrete in Odivelas dam. The magnitude of these displacements, measured before the first filling up, is dependent on grout's pressure utilized on foundation's treatment.

DISPLACEMENT OF EARTH SLOPES DUE TO SEISMIC SHOCKS. Sh.Prakash, P.Nandakumaran, V.Chandrasekaran, p.K.Mitra, B.V.K.Lavania /India/

The present day (1973) techniques of analytically determining the deformations of earth slopes due to earthquakes are indeed noteworthy steps in the right direction, but over-simplifications in these methods leave much to be desired. Model testing of dams with proper similitude gives more reliable results though some difficulty is experienced in proper model preparation.

At the School of Research and Training in Earthquake Engineering, model testing of Ramganga Saddle dam and Pandoh dam both 61 metre high have been successfully completed. Model analysis of the proposed Salal dam (100m high) is being done. The models of the dams are excited by impact of a heavy pendulum striking a shake table (4.88m x 2.74m). The acceleration of the table and the deformations of the model material are recorded. Another point worth considering is the shock to be simulated. Due to undertainties involved in the exact simulation of the materials, a more severe shock than would be required from similitude requirements is applied. For this both elastic and inelastic response characteristics due to expected ground motion are considered.

To throw more light on the draw-backs of the model testing, work on models of the same section with different scale ratios is being taken up. These models will be used to interpret the behaviour of each other and scaled up (or down) results will be compared with the observed ones.

At present studies on yield acceleration of cohesionless slopes which is a function of the nature of the ground motion to which it is subjected, are in progress. For a cohesionless slope, tested on a smaller shaking platform, it was found that a smaller value of yield acceleration results when subjected to steady state vibration than when simply tilted to cause failure. This has been interpreted as the effect of loss in strength when subjected to repeated loads. Attempts have been made in this study to know the deformed shape of the slope analytically and experimentally and compare the two. The variables considered are (i) slope angle (ii) number of cycles (iii) frequency and magnitude of exciting harmonic motion.

The analytical approach adopted is based on the yield acceleration concept proposed by Newmark (1965) and Goodman and Seed (1966). It is assumed that the sliding mass moves in the form of two wedges. The extent of movement is controlled by the strength-deformation characteristics of the slope material and the nature of the base motion. The deformed profiles are obtained considering rigid body movement of the sliding mass. The observed and computed deformed shapes for one of the cases are given in Fig.1.

The results of the experimental investigation on different cohesionless slopes subjected to steady state horizontal base motion reveal

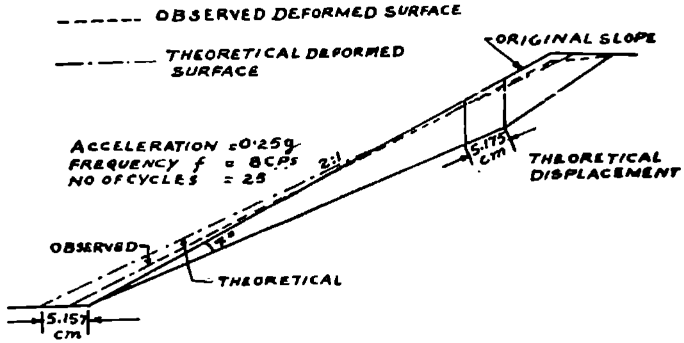

Fig.1. Comparison of theoretical and actual deformed shape of a slope

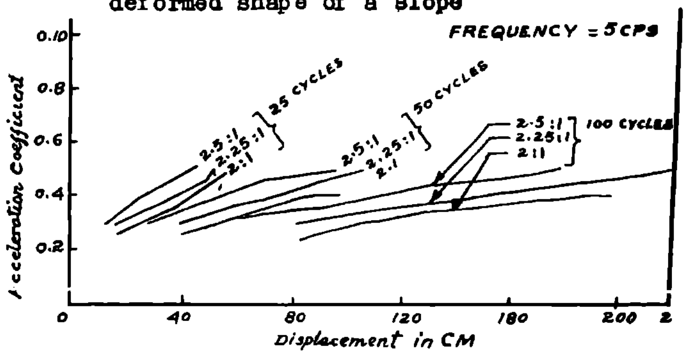

Fig.2. Effect of number of cycles on displacement

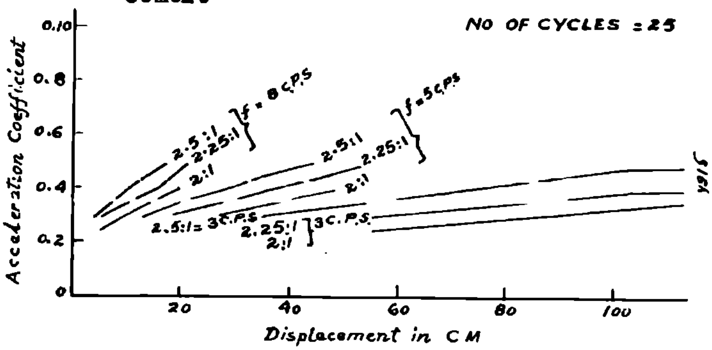

Fig.3. Effect of frequency on displacement

that the displacement increases with increase in the level of base acceleration, frequency and number of cycles. Some indication of these effects are shown in Fig.2 and Fig.3.

With the suggested technique it is shown that reasonable estimate can be made of the magnitude of repair work involved in embankments to be constructed in seismic zones.

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DESIGN OF EARTH AND ROCKFILL DAMS. Penman, A.D.M. /U.K./

The design of earth and rockfill dams to have a factor of safety against complete collapse gives little prediction of the amounts of movement which will occur in the actual structure. In addition to providing an adequate factor of safety, design should ensure that both during construction and during the impounding of water behind it, the deformations of the dam will be within acceptable limits.

Recent work of the Dams Section at the Building Research Station has been aimed at:

(a) developing a method of analysis to predict the movements of embankment dams, and

(b) measuring the actual movements which occur within the fill to compare with predicted movements and to obtain evidence of magnitudes of movement which are acceptable.

a) An analysis, using a finite element technique, has been developed to predict the movements from the results of large size laboratory tests on samples of fill material. An oedometer has been built to measure the deformation of samples of rockfill 1m diameter x 1/2m high under a stress of 3.2 N/mm^2 (320 tonnes/m^2). Tests have also been carried out in the Department of Civil Engineering at Imperial College, London, with samples 0.75m high x 0.31m diameter in triaxial compression and with oedometer samples 0.61m diameter.

We have been fortunate in being able to study the behaviour of both Scammonden Dam (70m high) and Llyn Brianne (90m high) currently the two highest dams in the UK.

b) Movements of a large number of steel plates buried in the two dams have been measured with a horizontal plate gauge, similar to that used at Gepatsch Dam in Austria. Movements of the surface of the dams, at the ends of the gauges, have been measured both by triangulation and by three-dimensional trilateration from an array of stable reference pillars placed some distance from the dams. A Kern DKM2A 1 second theodolite was used to measure horizontal and vertical angles from the pillars, and distances were measured by both the NPL Mekometer (accuracy 3 ppm $\pm$0.1mm) and by a Tellurometer MA 100 (accuracy 2 ppm $\pm$1.5mm). The accuracy achieved by the horizontal plate gauges in measuring both vertical and horizontal displacements of the steel plates was $\pm$2mm. The accuracy of both triangulation and three-dimensional trilateration in measuring vertical and horizontal movements of points on the surface of the dams was also $\pm$2mm, and the overall accuracy in measuring the movements of the buried plates in relation to the stable pillars is of the order of $\pm$3mm.

Agreement between the predicted and observed movements for the two dams has been most encouraging and has caused us to suggest that our method of analysis is sufficiently accurate for design purposes.

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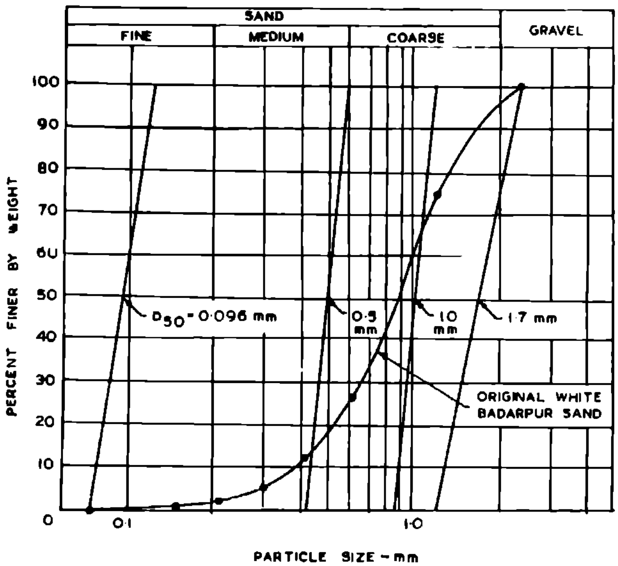
STRESS-STRAIN RELATIONSHIP FOR GRANULAR SOILS AT LOW AND HIGH CONFINING STRESSES.

T. Ramamurthy, V.K. Kanitkar, K. Prakash & C. Chawla (India).

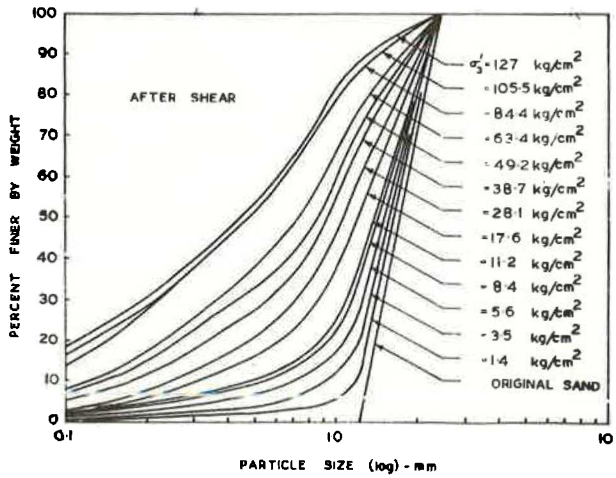
For analysing the stability and Predicting the behaviour of earth and rockfill dams using finite element technique, stress-strain relationships for soils and rockfill materials will have to be known under appropriate boundary stress or strain conditions on the representative materials. Testing of large rockfill materials consisting of rocks as large as $1m^3$ would require very elaborate and costly equipment. A central facility of testing such a material would not be adequate. In order to estimate the stress-strain relationship and other design data modelling of the field material, as suggested by Lowe (1964) for laboratory investigations appears to be very promising.

To study this aspect of modelling of rockfill materials four fractions having very nearly parallel gradings were obtained from White Badarpur sand quarried from Fatepur, Delhi. The grading curves for the original sand passing through B.S. Sieve No.7 and for the four fractions selected for the study are shown in Fig. 1. Two coarse and one medium sands having mean diameters (d50)



of 1.7 mm, 1.0 mm and 0.5 mm were obtained by sieving through successive sieves. A fine sand with mean diameter of 0.096 mm was obtained by grinding the original sand. The mean diameter of the coarser sand was 17 times that of the fine sand. Both the coarse sands were slightly angular than the other two sands. The tensile strength of particles decreased with increasing size.

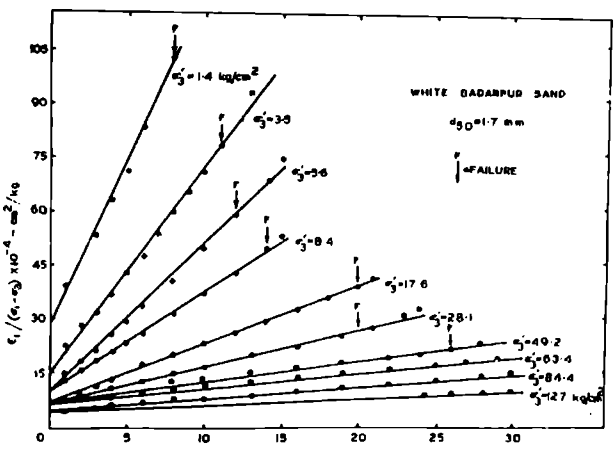
For conducting triaxial tests cylindrical specimens of 3.8 cm dia. and 7.5 cm ht. were prepared as dense as possible by rodding and tamping methods. Initial porosities for coarse sands varied from 40% to 42% while for the other two sands 44.5% to 46%. The rate of strain was 0.125 mm/mt. These sands were tested under full saturation in isotropic consolidation and consolidated drained condition in triaxial compression upto a maximum confining stress (σ_3) of 127 kg/cm². The gradation analyses of these sands before and after shear suggested considerable crushing of particles. Fig.2 shows such changes for the coarser sand.



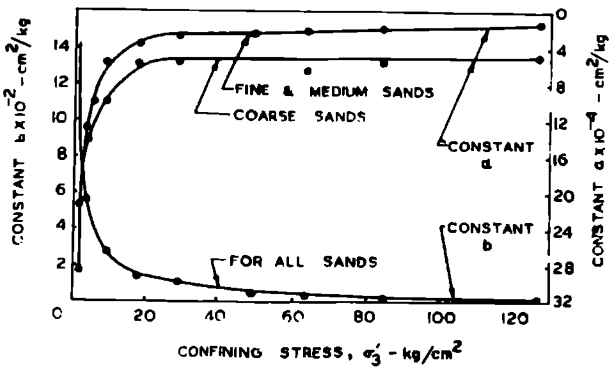
The magnitude of crushing expressed as the percentage passing through the sieve on which the sample was initially 100% retained, increased with increasing particle size. The magnitude of crushing in the coarser sand was about 90% while in the fine sand about 40% when these sands were consolidated and sheared upto 30% axial strain under an effective confining stress of 127 kg/cm². The uniformity coefficient of coarser sand changed from 1.3 to about 11 and that of the fine sand from 1.3 to 3 at the end of shearing. The gradings produced at the end of shearing were markedly different from one sand to the other. Although the magnitude of crushing was larger in coarser sand but the specific surface area produced in all the sands were identical for similar energies given to the specimens during shear (Ramamurthy et al 1973).

During isotropic consolidation, the volumetric strains due to recoverable and non-recoverable compressions of the specimens after separating the membrane penetration increased with increasing particle size (Ramamurthy & Kanitkar 1972). The effective stress ratios at failure varied between 6.3 and 5.6 for σ_3 = 1.4 kg/cm², between 3.35 and 3.65 for σ_3 = 38 kg/cm² and between 3.3 and 3.4 for σ_3 = 127 kg/cm². All the four sands indicated effective angle of shearing resistance at failure within 44° to 47° at low confining stresses. The coarse sands showed slightly higher values. The coarser sand mobilized lower angle of shearing resistance with axial strain than the fine sand. For 2% and 5% axial strains coarser sand mobilized 10° and 16° while the fine sand mobilized 17° and 23° respectively. Since the coarser sand was crushing more than the fine sand some of the axial strain was absorbed in crushing similar to plastic strain. The effective angle of shearing resistance at failure for all the sands at high confining stresses varied between 33° and 32°, inspite of the differences in particle size, particle strength and magnitude of crushing. Generally failure strains increased with increasing particle size, confining stress and magnitude of crushing. The strains at failure were as large as 30%. The coarser sands exhibited large volumetric contraction (about 19%) than fine sand (about 12%) under high confining stresses. The coarser sand showed lateral contraction of the specimen (-ve poisson's ratio) during shear for confining stress greater than 63.4 kg/cm² upto axial strains varying from 5% to 10%. The dilatancy rates at failure were about 1.8 at σ_3 = 1.4 kg/cm² and decreased rapidly upto σ_3 = 28 kg/cm² and thereafter remained constant having values between 0.7 and 0.85. Some of the above findings were also observed by Marachi et al (1972).

The stress-strain curves for all the four sands tested upto 127 kg/cm² confining stress were found to be hyperbolic in shape and could be represented by $\epsilon_1/(\sigma_1-\sigma_3) = a + b\epsilon_1$. Such a constitutive law was proposed and verified by Kondner and Zelasko (1963) for granular soils sheared at low stresses. In addition to the different forms of this relationship suggested by Hansen (1963), numerous other relations were tried but only hyperbolic representation of stress-strain relationship was found to define reasonably a simple constitutive law for granular soils subjected even to high stresses. Fig. 3 shows such a representation for coarser sand.



The values of constants 'a' and 'b' decreased rapidly for the four sands with increasing confining stress upto 12 kg/cm² and later on the decrease was not significant, Fig. 4.



The constant 'a' is the inverse of initial tangent modulus is slightly effected by the particle strength. Coarse sands have higher values of 'a' than the other sands since they were weaker. The values of 'b' were almost same for all the four sands for similar confining stress, i.e. the ultimate deviator stress is uneffected by size, shape and tensile strength of particles.

These studies revealed that the constitutive law, peak effective stress-ratio, ultimate deviator stress, and dilatancy rate at failure are not likely to be influenced by modelling of the materials but mobilization of strength with strain, axial and volumetric strains at failure, the magnitude of crushing and elastic and plastic compressions during isotropic consolidation will be influenced; initial tangent modulus and poisson's ratio are also likely to be effected to same extent. In the absence of a generalized constitutive law hyperbolic stress-strain relationship may be adopted to predict the behaviour of granular materials even if considerable crushing occurs under high stresses.

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STRESS-STRAIN ANALYSIS AND STABILITY OF EARTH AND ROCKFILL DAMS UNDER STATIC AND DYNAMIC LOADS. L.N.Rasskazov, M.V.Vittenberg, D.V.Kiranov, M.N.Voloknova, I.S.Klein /USSR/

Earth and Rockfill Dams are complex three-dimensional Structures made of materials having non-linear, elastic-plastic properties. At VODGEO Institute the solution of problems connected with stress-strain state in earth and rockfill dams has become possible with the aid of finite element method similar to the one described by O.C.Zienkiewicz, Y.K.Cheng and R.W.Clough which is used alongside with the method of local variations developed by F.L.Chernousko and N.B.Bonchuck.

In solving non-linear, plastic-elastic problems it has become necessary to work out a new definition of the matrix properties of soils, to show that the iteration procedure is accurate enough as well as to consider the plastic properties of soils by a new method.

The stress-strain state and stability of a rock-fill dam with a central vertical core may be solved by separation of a cross-section into 306 elements. The soil characteristics have been carefully examined in a triaxial apparatus and the test data are presented in Fig. 1 (loam has water content =13,5 per cent and γ soil skeleton = 1,7 g/cm^3).

Coarse gravel and limestone showed correspondent characteristics. It is only natural that the modulus of deformation for rock-fill materials is 10 to 15 times higher then for cohesive soils. Poisson's ratio for rockfill materials is two times lower for rockfill materials then for pebble-gravel materials though the stress state was identical.

Fig.2 shows the distribution of stability number K_3= along the cross-section of a dam (The instant process of construction and plain strain are assumed), φ assumed and φ natural being derived from Mohr's stability analysis.

The existanse of ultimate zones inside the upstream part and the core (K_3 =I) of a dam is not dangerous for the displacement vectors are directed downstream.

It must be pointed out that if $E=E(t)$ and $\mu=\mu(t)$ we can incorporate creep effects into analysis.

The dynamic analysis of dams was also based on the finite element method for both one phase and multy phases vibration. All the procedures were associated with the mass matrix equivalent to the mass distributed throughout the element. Also the integration of the response of a structural system to the transient damping force was carried on in a variety of ways.

The aim of the dynamic problem is to find out the residual displacements inside the dam after earthquake being over.

To assist in solving the problems mentioned above it was necessary to modify a computational technique and to verify the results with the tests in triaxial apparatus under vibration.

It was required that the residual displacement should be determined and compared with both dynamic and static characteristics.

Three-dimensional problem could be treated with the method of local variations, founded on the principle of minimum energy which may be described as

$$\Pi = \int_V \sigma_{ij}\,\delta e_{ij}\,dV - \int_V \rho\,\delta f\,dV - \int_S R\,\delta u\,dS \quad (1)$$

in this σ_{ij} and e_{ij} are stress and strain components in an element having V value

S- unit weight of soil
f- displacements inside a certain point of an element
R- outer distributed forces
u- displacements of the applied outer forces
s- the region of the application of outer forces

The minimization procedure requires node to node displacement while finding the minimum "u" along xyz axis with the step value being h_1-h_{10}.

The displacement components e_{ij} have to be defined by dividing the region into tetrahedrons.

With this method the problem can be solved for constructions in stages inspite of the stress-strain relationship is a very complicated one.

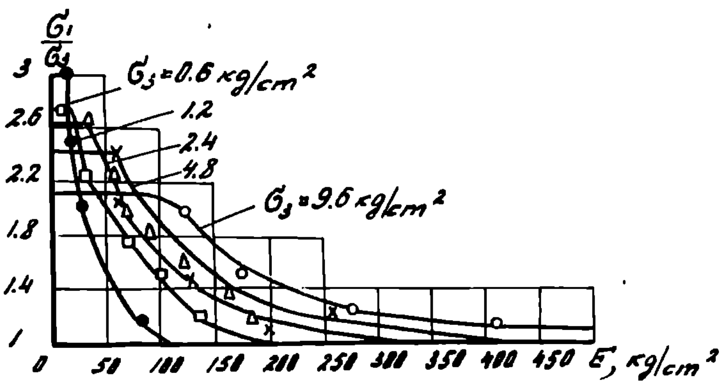


Fig.1

Expressions $T=T(e,\Gamma)$ and $\sigma=\sigma(e,\Gamma)$ were found experimentally where σ, e, T, Γ are the first stress and strain matrix invariants respectively and the second invariants of stress and strain deviation (S_{ij} and E_{ij}). The strains are assumed to be linear in soil during unloading.

Beyond the ultimate load the yielding was expressed in terms of Prandtl-Reiss's condition.

Thus,

$$K_3 = \frac{U_c + S\sigma de}{\int S_{ij}\, d\mathcal{E}_{ij}}$$

where U_o- cohesion energy of the material this expression the state of soils is easily obtained.

The main aim of the further investigations is to find out the summarized stress-strain relationship.

In order to evaluate the stability of earth and rockfill structuress more reliably using the stress-strain state.

The progress in dynamics may be obtained by proper choosing of design accelerograms and proper analysing of structure stability.

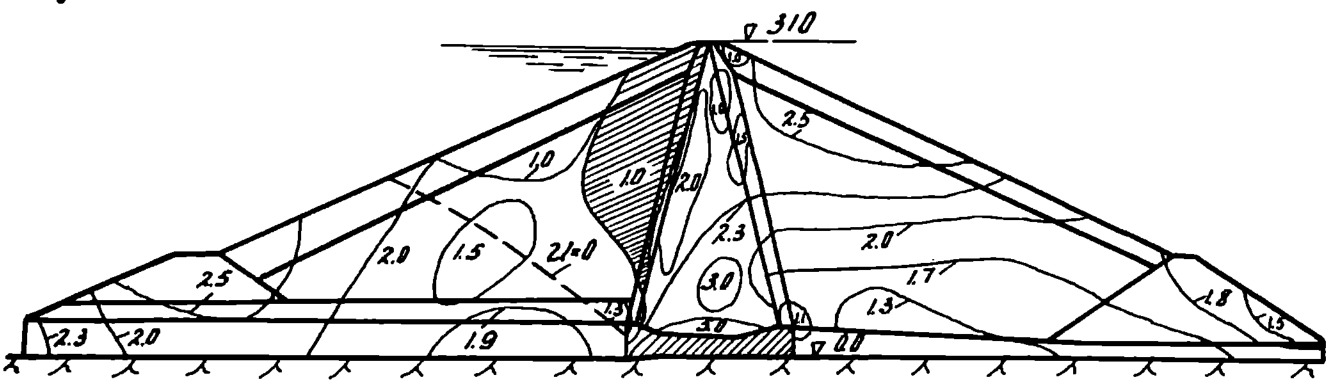


Fig. 2

L'INFLUENCE DE L'ETAT DES CONTRAINTES SUR L'ANGLE DE FROTTEMENT INTERNE DANS LES SOLS GRAVIER. A.P.Rijenko (URSS)

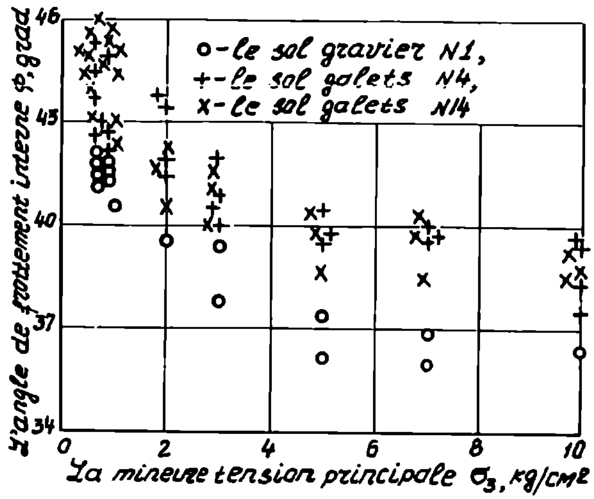
Parmi plusieurs facteurs, qui influencent sur la valeur de l'angle de frottement interne ϕ , il faut souligner celle, qui concerne l'état des contraintes du sol. Pour développer la correlation $\phi = f(\sigma)$ l'auteur a essayé sur le grand appareil triaxial hydraulique TzNIIS les trois espèces des sols graviers avec les dimensions des particules 10; 40 et 60 mm, les échantillons avaient le diamètre 30 cm et la hauteur 70 cm (A.P.Rijenko, 1968). On a varié la pression latérale sur l'échantillon de 0,6 à 10 kg/cm^2 ce qui correspond au changement de la pression normal de 1 à 17 kg/cm^2. La compacité des sols était moyenne. Poids unitaire de squelette du sol et la porosité sont conformément égales 1,60; 1,83; 2,00 t/m^3 et 39,6; 31,0; 26,7%. A la figure sont representées les valeurs obtenues comme la relation entre l'angle de frottement interne ϕ et la mineure tension principale σ_3 . D'après ces données, à l'aide de la machine digitale, on a trouvé les équations correlatives comme des paraboles carrées ou cubiques. On a obtenu pour chaque espèce des sols étudié quatre équations suivantes

$$\phi = a + b\sigma_3 + c\sigma_3^2 , \quad (1)$$
$$\sigma_3 = a + b\phi + c\phi^2 , \quad (2)$$
$$\phi = a + b\sigma_3 + c\sigma_3^2 + d\sigma_3^3 , \quad (3)$$
$$\sigma_3 = a + b\phi + c\phi^2 + d\phi^3 . \quad (4)$$

On trouve la meilleur coïncidence des données experimentaux pour la courbe (3).Ecart carré moyen des valeurs d'essais et ceux calculées d'après ces équations était dans les limites 0,203-0,294, l'étriotesse de la liaison relative changeait de 0,968 à 0,888. L'équation (3) pour chaque espèce du sol gravier examiné se distingue essentiellement par la valeur du membre premier. En le désignant comme ϕ on peut remplacer les trois équations par une seule équation corrélative suivante

$$\phi_\sigma = \phi - 2{,}18\sigma_3 + 0{,}26\sigma_3^2 - 0{,}01\sigma_3^3 , \quad (5)$$

où ϕ_σ - la moyenne de l'angle de frottement interne.
On a verifié la possibilité de remplacer les trois équations (3) par une équation (5) en confrontant les valeurs ϕ_σ, calculées selon ces équations pour les valeurs differentes de σ_3. Cette confrontation a montré, que l'erreur maximale, qui resulte de cette remplacement, est 4,5%; ce qui ne dépasse pas l'exactitude de détermination des caractéristiques mechaniques du sol obtenues à l'aide de l'appareil triaxial.



La grandeur σ_3 on estime souvent dans la méchanique des sols comme le produit du poids unitaire du sol sur la profondeur de la fondation, c'est-à-dire γD. Dans ce cas on peut représenter l'équation (5) comme

$$\phi_\sigma = \phi - 2{,}18\gamma D + 0{,}26(\gamma D)^2 - 0{,}01(\gamma D)^3 \quad (6)$$

Pour l'utilisation pratique on la reduit à la formule suivante

$$\phi_\sigma = \phi K K_k , \quad (7)$$

où ϕ_σ - valeur corrigée de l'angle de frottement interne;
ϕ - valeur moyenne de l'angle de frottement interne, déterminée à l'aide de l'appareil triaxial lors la prèssion latérale est 0,6 kg/cm^2; avec cela le diamètre de l'échantillon dépasse le moins 5 fois la dimension la plus grande des particules de gravillon;
K - coefficient de l'homogénéité du sol, tenant compte de la possibilité de divergence des valeurs obtenues;
K_k - coefficient correctif, qui estime l'influence de l'état des contraintes du sol sur la valeur ϕ; on le trouve par le tableau suivant la valeur du poids mort du sol au niveau de la base de fondation.
La relation, obtenue lors des essais (7) permet tenir compte du changement de l'angle de frottement interne des sols graviers suivant l'état des contraintes à la base de fondation, c'est-à-dire on introduit dans les formules de calcul la valeur corrigée de caracteristique fondamental calculé.

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Valeur du poids mort du sol au niveau de la base de la fondation $\gamma D^{x)}$, t/m^2	6	10	20	30	40	50	60	70	80	90
Valeur du coefficient K_k	1	0,985	0,951	0,925	0,909	0,896	0,890	0,886	0,884	0,882

x) Le poids unitaire du sol est estimé sans l'effet hydrostatique d'eau.

LIME FIXATION OF DISPERSIVE CLAY, MERRIMU DAM, AUSTRALIA. M.G. Speedie and V.S. Michels (Australia).

Introduction. Dispersive clay soils extend over substantial areas of the State of Victoria. Being readily erodible, they present a piping hazard in the construction of homogeneous and clay-core dams required to store low-salinity water. Because alternative materials are rarely available, such clays commonly require treatment to reduce their erodibility.

Merrimu Dam, an earth-and-rockfill structure 37 m high and 128 m long, is located on Coimadai Creek in southern Victoria, and has a central impervious core. As the clay for the core was dispersive, the downstream 3.7 m of its horizontal width was treated with hydrated lime. The resulting partial replacement of sodium ions with calcium, converts the potentially erosion-prone clay to a more stable material, but with a lower density, higher optimum water content, and greater permeability.

As piping due to seepage can start only at the interface between the impervious zone and the downstream filter, it is the downstream face that should be treated: This prevents initiation of a pipe, and simultaneously maintains the minimum overall seepage gradient over the whole width of the impervious zone. On the other hand, if the upstream face were treated, this would not hinder the initiation of a pipe at the downstream face. Such a pipe could form as in an untreated dam and progress upstream to the treated zone. Then the whole hydraulic head would be acting over the limited width of the treated zone, subjecting it to a much more severe gradient and a greater possibility of failure than for the case of downstream treatment.

Clay Properties. The core material, obtained from elevated slopes of Recent and Tertiary hillwash, was a reddish brown, sandy or silty clay of medium to high plasticity, with the following properties:

Classification, MI/CI to CH;

γ_s = 2.71 to 2.78;
k = 0.1×10^{-6} cm/s or less;

I_P = 24 to 70%;

ϕ' = $1\frac{1}{2}$ to $21\frac{1}{2}^{\circ}$, design value 6°;

c' = 48 to 21 kPa, design value 46 kPa;

w optimum for 40-blow Proctor test, 16.8 to 23.6%;

γ_d = 1.80 to 1.59 tonnes/m^3.

Following treatment with 2% of hydrated lime:

w optimum, 23 to 29%;

γ_d = 1.57 to 1.46 tonnes/m^3;
k = 0.5 to 2.4×10^{-6} cm/s.

Dispersion Tests. Two types are normally performed: a) Crumb Test (Emerson and Northcote, 1964); b) Quantitative Test (Reilly, 1968), whereby the soil is dispersed in hydrometer jars, with and without a deflocculant, and the resulting suspensions are sampled 8 cm below the surface after one- and three-day settling periods. The weight of solids in suspension without deflocculation, expressed as a percentage of that obtained with a deflocculant, is reported as the Percentage Dispersion (P.D.). This test was devised by Mr. L.A. Reilly, and has been used in the laboratories of the S.R.&W.S.C., Victoria, since 1964.

Clay-deflocculation piping failures in the banks of earthen channels and urban storage basins in Victoria, have occurred in soils with P.D. values exceeding 70% and 50% at one and three days, respectively. At Merrimu, of 48 tests on borrow-pit samples, two-thirds had P.D. values exceeding both these limits. However, of the 14 tests on compacted samples from the untreated clay placed in the core, the corresponding proportion was only one-seventh for both settling periods, possibly due to the slightly greater salinity of these samples.

Specification for Hydrated Lime. Mechanical Analysis: Passing B.S. sieve No. 52, 99% min; No. 200, 84% min. Chemical Analysis: Loss on ignition, 22±2%; CaO, 68±4%; MgO, 5±2%.

Field Procedure. The lime was spread uniformly in the borrow-pit at the rate of 2% of the clay γ_d, adding water to obtain optimum w, and premixed together by rotary hoe. The mix was stockpiled in the borrow-pit, well ahead of placement, to avoid any interruption of work on the dam, and protected against moisture changes.

Costs. The extra cost of lime fixation per cubic metre, i.e. in excess of the normal overall clay cost, was $1.05 for supply and spreading, plus $0.80 for mixing and wetting, totalling about $2.00/m^3 after allowing for additional handling.

Performance of Dam. The dam was completed in 1969. The reservoir half filled in 1970, and overflowed in 1971. No seepages through or under the dam have been observed to November, 1972.

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DESIGN OF FILTER THICKNESS BASED ON THE APPLICATION OF QUEUEING THEORY.

V. Thanikachalam (India), R. Sakthivadivel (India)

Protective filters which form an integral part of the high and massive earth and rockfill dams demand large quantities of filter materials, and under certain conditions, have created a need for using crushed stones to economise the cost of filter construction. Although filters have been in use for more than half a century, as of now, no rational design procedure for protective filters is available.

The design of filters mainly consists of choosing the gradation of the filter material, its breadth and thickness. The filter materials chosen, has to satisfy two contradicting requirements namely that it should be more pervious than the base soil to allow freely the seeping water without causing excessive head loss; and at the same time it should be able to retain the grains of base material and arrest piping (4). These requirements are satisfied partly by the grain size criteria which gives the gradation of the filter material for a given cohesionless base material. The width of a filter is generally taken as multiples of the size of the earth moving equipments used for construction (5). If the grain size criteria is completely satisfied, then only a nominal thickness is sufficient to account for the segregation of the filter materials and/or the settlement of earth mass. On the other hand, if the required gradation of the filter material is not available, then for the use of such materials, thikness of a filter becomes an important parameter and requires careful consideration.

Presently the thickness of a filter is determined from two approaches: 1. Seepage analysis which provides adequate cross-sectional area to drain the seeping water without causing excessive seepage stress; 2. Probability analysis which considers the random movement of the base particles in the filter pores and provides the required thickness to arrest the washing out of the base soil from the filter (3).

In the present investigation, the random movement of base particles and its clogging of a filter pore channel are treated as a stochastic process. A theory is formulated based on the 'Queueing Process' to determine the filter thickness for a given base and filter materials. The application of the queueing process is based on the assumption that the washing out of the base particles will be stopped, only when the leading particle in the queue is arrested from further movement. In this theory, the arrival of base particles into the filter is characterised by Poisson rate λ. The clogging of the pores of a filter by base soil is assumed to be exponentially distributed with depth. The required solution of the problem is then to determine the number of particles in the queue when the system reaches equlibrium. Equilibrium of the system refers to the clogging of a pore channel by the leading particle at time t, tending to infinity. Considering the probability of n base particles in the system at time $(t + \Delta t)$, time dependent equations for 'n' number of particles are obtained. The steady-state solution is considered to determine the mean and the variance of the number of particles in the queue. The thickness of the filter is determined by assuming that a finite number of base particles are held by each layer which is represented by the average size of the filter particle. From the number of layers computed, the filter thickness is obtained.

The computed thickness of a filter based on the queueing theory compares well with that obtained by Markov Chain (3). The main advantage of queueing theory is that a knowledge of the pore size distribution, the flow velocity are indirectly taken into account through the two parameters and whereas in Markov Chain, the actual pore size distribution is required which is difficult to obtain in practice. From the solutions obtained by queueing theory, it is noted that by increasing the thickness beyond a certain range does not marked by commensurate increase in the efficiency of trapping the base particles.

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INVESTIGATIONS OF ROCKFILL DAM CORE CRACKING BY CENTRIFUGAL MODELLING. V.I./utsel, V.I.Shcherbina. USSR.

Centrifugal modelling is thought to be the most promising method of studying the rockfill dam core cracking. The main advantage of this method, gaining lately a wide recognition, is that it enables the geometrically similar structure model, made of the prototype material, to be studied under the stresses conditions similar to that of the prototype structure as regards the magnitude and the pattern of stress distribution. The fact that the model is loaded only by internal (volumetric) forces without using any external loading devices, as occurs in real structures, is of particular value.

Cracking was tested on a model core of a rockfill dam 66.0m in height, located in a symmetric rock canyon. In the first stage the longitudinal core section was studied, influence of shells was not considered. For model tests soil was used with the moisture contents: $W_T = 19\%$, $W_p = 15\%$, $W_n = 4$.

The soil placed in the model had the optimum moisture content W=11% and density $\gamma = 2.05-2.07$ g/cm^3. It was compacted by 2cm layers with a falling ram having a rectangular load plate (S=48cm^2) with the weight of 2.5 kg. During the model preparation the soil density and moisture content have been controlled by sampling, which was also performed after completing the test.

The core model was made in a special form with transparent side walls. The canyon wall slopes of 40° were made of massive organic glass. A double layer of polyethylene film interspaced with vaseline was provided to prevent the side wall friction. The model (geometric scale 1:200) was tested in the centrifugal force field at acceleration of 200g, which was adequate for achieving stresses identic to those existing in the full-scale structure.

Both the core crest vertical and horizontal deformations were measured with the help of specially developed transducers. The conducted tests revealed that in the upper part of the dam core adjacent to the canyon walls tensile zones were developed, while in the core centre a compressive zone appeared. It tensile strain value exceeds the ultimate one for this type of soil, then brittle fracture (cracking) occurs. Model cracks were symmetric to the canyon axis, and their location was rigorously following the points of maximum tensile strains (Fig. a, б)

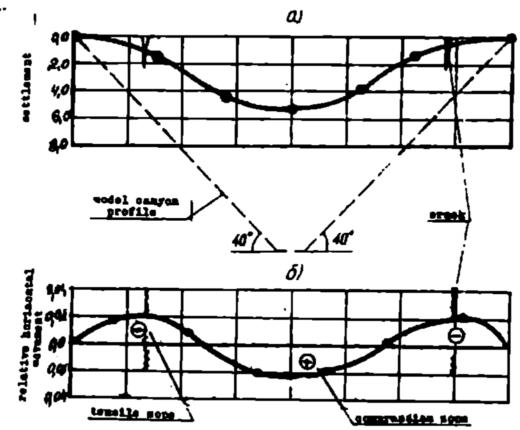

Fig. a) Vertical deformations of core crest. b) Horizontal displacements

The cracks were developed at the distance of (0.2–0.3)B from the canyon walls (here B= crest width of canyon), their depth was 45–50mm which after recalculation for the full-scale dam yielded the value of 9–10m. Detection of the moment when the cracks appeared on the model crest permitted to determine the value of the ultimate extensibility of soil. It should be noted that the general pattern of distribution of tensile and compressive zones, crack formation and their depth are in full conformity with the field results for a number of dams (Rector Creek, Peroucha, Gepatsch, El Infiernillo).

Centrifugal model studies helped to reveal the deformation characteristics of the core material. On the basis of settlements measured in different sections with various heights mean deformation moduluses were determined. Moduluses distribution over the depth is thought to be non-linear: in the upper zone moduluses smoothly increase with depth from 100 to 300 kg/cm^2, and in the lower part of the model they sharply increase and at the depth of 0.8H reach 800 kg/cm^2. Indicative is that these moduluses correspond to the construction period of a rapidly built structure.

CONCLUSIONS

Centrifugal model testing for the first time permitted to determine in laboratory the behaviour of core prototype soils at stressed-strain state, closely approximating the field one. Conditions and zones of crack formation as well as the depth of their distribution were determined on a core model in a symmetriccanyon. Study of deformation characteristics of soil material is supposed very promising.

INFLUENCE DU DRAINAGE SUR LA STABILITE DES DIGUES DES BASSINS DE DECANTATION.

J.Wencewicz /Pologne/

Les cendres, un materiau de dechet des centrales thermique sont deposees hydrauliquement dans des bassins de decantation, ces bassins sont entoures de digues en terre ou en cendre. Certains accidents des digues ont attire notre attantion sur le probleme de leur stabilite. Et par ce fait on a donc decide d'etudier:

- le fonctionnement des drainages in situ
- les manieres de construction des bassins de decantation des diverses centrales thermiques
- les proprietes physico-chimiques et mecaniques des cendres en tant que materiau pour la formation des digues
- La filtration sur le modele d'analogie visqueuse.

Les bassins de decantation pris en consideration sont tres differencies au point de vue superficie /50 - 380/x10^3 m^2, volume /240-2000/x10^{3}m^3, hauter d'endiguement et par la solution de la construction des drainages. Toutes les digues ont ete sureleves par des remblais de la II-ieme et III-ieme etape construits en cendre.

Les cendres par elles memes ont des proprietes physico-chimiques bien variable /tableau 1/, influencees par divers facteurs notamment:
- par la sort de charbon brule et sa granulometrie
- par la distance entre le deversoir de l'eau decantee et le jet du melange eau-cendre.

Tableau 1.

Propriete physico-chimiques des cendres

Proprietes	Centrale thermique No 1	2	3
densite g/cm^3	2,15-2,24	1,70-3,13	1,95-2,83
densire apparente g/cm^3	1,08-1,28	1,10-1,50	0,88-1,13
densite ap.seche g/cm^3	0,71-1,01	0,80-1,05	0,78-0,90
humudite %	29,2-47,6	27,0-50,0	12,6-25,2
coeficient de compactage	0,66-0,88	0,70-0,90	0,82-0,95
coefficient de filtration cm/s	-	1,65x10^{-2} 3,45-10^{-3}	10x10^{-3} 9,9x10^{-4}

Les observation in situ ont demontre que le fonctionement des systemes de drainage n'etait pas partout parfait.
Dans certains cas ou l'on a constate des glissements du talus les drainages n'existaient pas.

Les etudes sur modele a l'analogie visqueuse de la fitration a travers une digue deja construite ont verifie une conclusion precedente, qu'un emplacement du drainage au pied du talus donne des effects insuffisants. Un drainage de superficie pose sur le talus assure l'absence des phenomenes defavorables pour la stabilite de la digue tels que suintements, glissements etc. Ce type de drainage dont la construction, l'expoitation et les travaux de restauration eventuele sont simples, commodes et protege aussi le talus contre l'erosion. L'exigeance de quantites considerables de materiau fultrant peut etre admis come un desavantage peu signifiant de ce type.

Une etendus relativement faible des etudes accomplies ne permet pas formuler des conclusions preliminaires concernant l'applicabilite des certains types de drainage.

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COMPARISON OF THE DEFORMATION OF THE TRESNA DAM, OBSERVED AND COMPUTED BY MEANS OF THE FINITE ELEMENT METHOD. Wojciech Wolski (Poland)

The 38 m high Tresna Dam /Fig.No1/ on the Sola River was constructed between 1962 and 1964. The earth and rockfill type dam has a central core made of silty loam with moisture content considerably exceeded the Proctor Standart optimum /$w_n=1,2 \div 1,4\ w_{opt}$/. Owing to the significant compressibility of the core material, an unusual deformations of the dam was anticipated. However investigations and observations made in the course of construction and during five years of exploitation, has shown that the deformation are in usually encountered limits. The observed deformations were compared with those computed by means of finite element method .

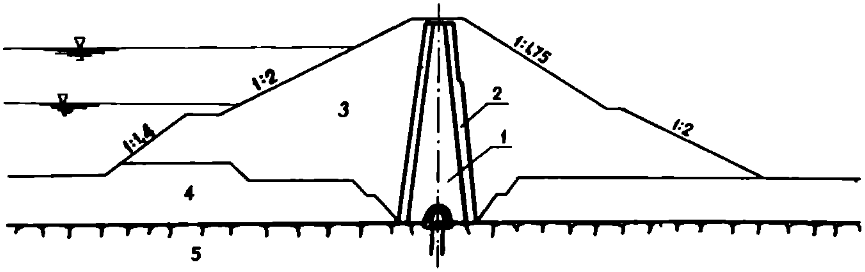

FIG.1. CROSSECTION OF THE TRESNA DAM

1. impervions core, 2. transitive filters, 3.shell, 4. alluvium, 5..bedrock

The comparison has demonstrated good agreement between theoretical results and observations.

The displacements and state of stresses in transvers section of the Tresna Dam was computed by means of finite element method /4/, using the quadrangular elements. The following assumption were made:

- plane strain conditions exist ,
- the materials of the dam were linearly elastic,
- the foundation of the dam were rigid,owing to the high modulus of compressibility of the bedrock.

Computation were made for two cases:
I- end of construction /empty reservoir/,
II- maximum storage pool. In case I the only load considered was the weight of the embankment; in case II the water pressure from the pool acting on the upstream slope of the core were assumed, and a submerged soil weight of upstream shell was used. The results of the computations for both cases enabled to evaluate the displacements of the dam caused by pressure of the water. The comparison of the horisontal displacements of the crest of the dam, evaluated by the above mentioned method and displacements of the benchmarks located on the crest of the dam,surveyed using precision method, demonstrated a fairly good agreement /Fig.2/. It could be therefore concluded that it is possible to evaluate the horisontal displacements of the dam by means of the finite element method. However it should be stressed, that on account of the neglect of the time factor, it is only possible to evaluate final displacements

The finite element method was also used to check if there are possibilities of the development of the tension cracks in the core of Tresna Dam. It was found that minimum compressive stress occur in upper portion of the core in plains inclined at 45° to the horisontal line /Fig.No3/, - tensile stresses did not occur ,it could be therefore assumed, that there is no conditions for development of the cracks. This conclusion was confirmed by the borings performed in the dam core /2/. Undistrubed soil samples taken from the core, demonstrated the lower water content and higher density in comparison to those estimated during construction, what proves that consolidation takes place; also no traces of cracks or fissures was encountered in undisturbed samples.

It should be pointed out, that location of the lowest stresses in the core of Tresna Dam coincided with the observed cracks zones in the Hyttejuvet Dam /1/ and Bolderhead Dam /3/.

The results of observations and computations presented in this comunique proved that by application of finite element method it is possible to predict the displacements of the parts of the dam and to locate zones within dam cores where tensions cracks are likely to develop .

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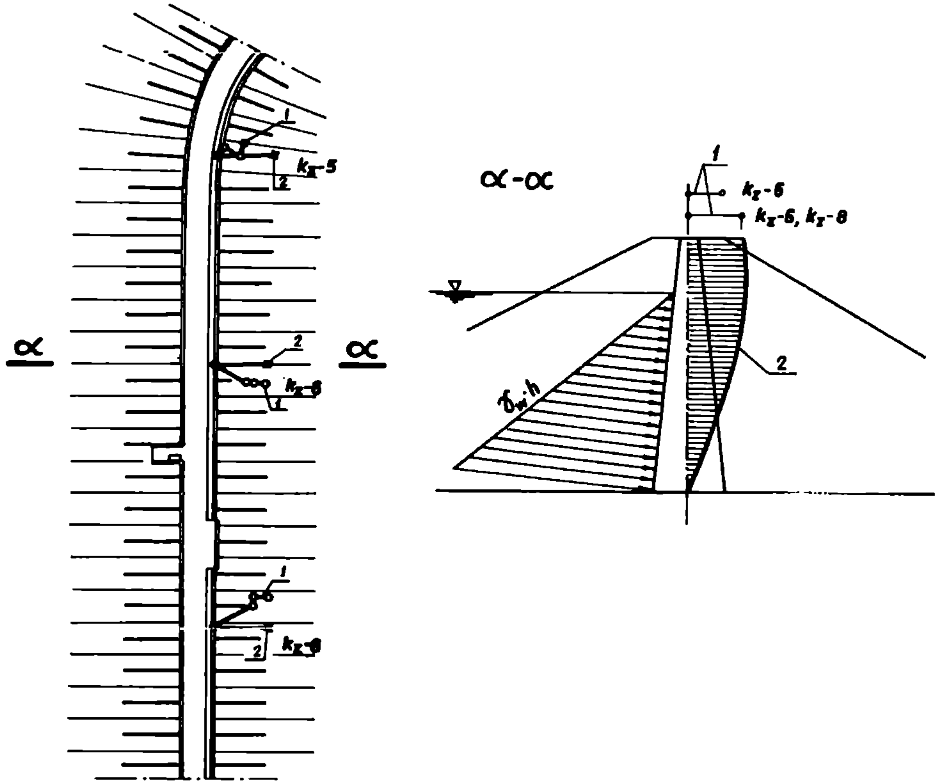

FIG.2 COMPARISON OF THE DISPLACEMENTS SURVEYED, AND COMPUTED
1. displacements surveyed, 2. displacements computed

a) b)

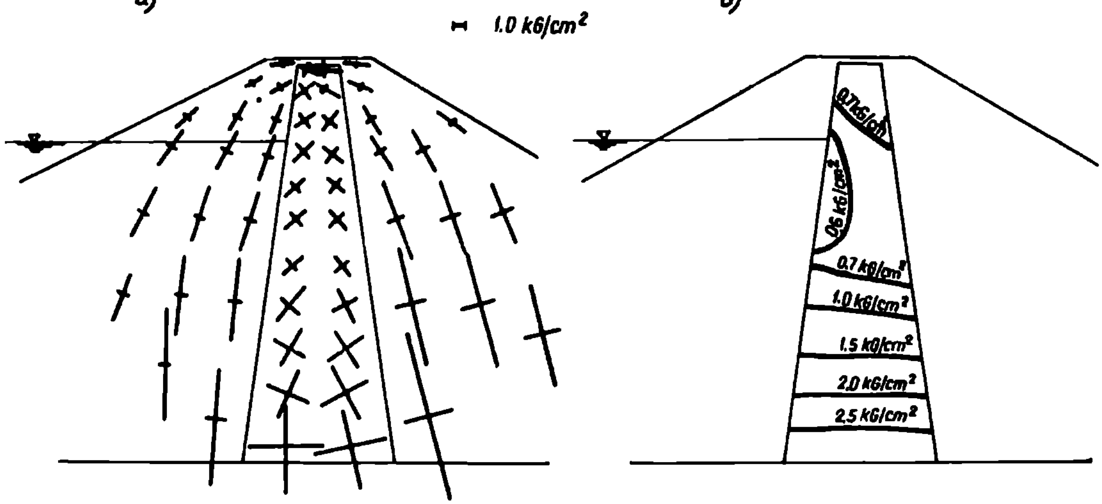

FIG.3. RESULTS OF THE ANALYSIS WITH FINITE ELEMENT METHOD
a. principal stresses; b. isolines of compressive stresses

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The 38 m high Tresna Dam on the Sola River was constructed between 1962 and 1964. The earth and rockfill type dam has a central core made of silty loam with moisture content considerably exceeded the Proctor Standart optimum /w_n = 1,2 ÷ 1, 4 w_{opt} /. Owing to the significant compressibility of the core material, an unusual deformations of the dam was anticipated. However investigations and observations made in the course of construction and during the five years of exploitation, has shown that the deformation are in usually encoutered limits.

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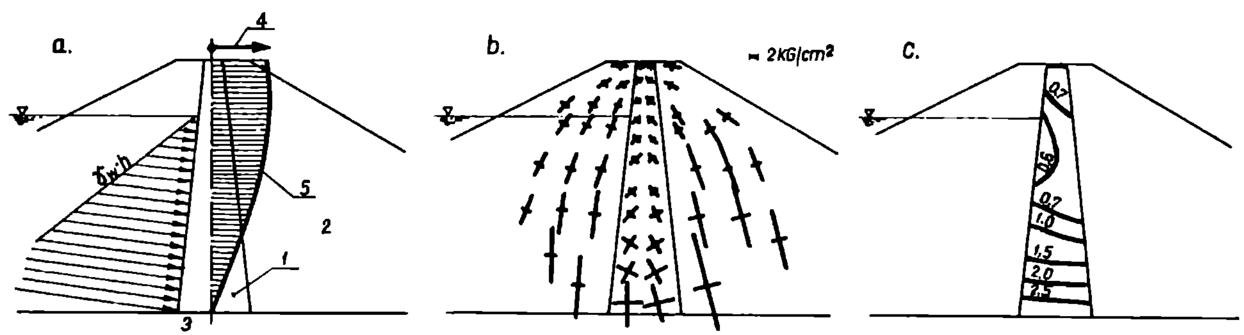

Fig.1. Results of the analysis with finite element method: a/ comparison of the displacement surveyed and computed / 1 - impervions core, 2 - shell, 3 - bedrock, 4 - displacements surveyed, 5 - displecement computed/, b/ principal stresses, c/ izolines of compresive stresses.