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CENTRIFUGE MODELLING STUDIES ASSOCIATED WITH DESIGN AND CONSTRUCTION OF EARTH DAMS

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Many problems that arise during construction and design of earth hydroengineering complexes, including those such as estimating the bearing capacity of an inhomogeneous base, determining the lateral pressure of the soil, determining the criterion of crack formation in the cores of dams, estimating the stability of the slopes of structures and so on, cannot be resolved confidently when conducting ordinary model experiments with full-scale soils. The main reason for this is that the stresses due to the natural weight in the models are over

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered) considerably less than those in real structures: at the same time, as shown by investigations of many authors, the deformation and strength properties of soils under small and large stresses may differ considerably and not only quantitatively but qualitatively as well. To overcome this difficulty permits the use of the centrifuge modelling method in investigations, the main advantage of which is the possibility of investigating relatively small models made of full-scale soil under stresses similar to full-scale in absolute value and by the nature of distribution. This report goes into detail concerning this modelling technique.

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CENTRIFUGE MODELLING STUDIES ASSOCIATED WITH DESIGN AND CONSTRUCTION OF EARTH DAMS

V. I. Shcherbina

CENTRIFUGE MODELLING STUDIES ASSOCIATED WITH DESIGN AND CONSTRUCTION OF EARTH DAMS

Moscow SOVETSKO-AMERIKANSKIY RABOCHIY SEMINAR "TEKHNOLOGIYA STROITEL'STVA SOORUZHENIY V USLOVIYAKH KHOLODNOGO KLIMATA," in Russian 16-28 May 1977 pp 1-18

[Article by V. I. Shcherbina, All-Union Planning, Surveying and Scientific Research Institute imeni S. Ya. Zhuk]

[Text] Many problems that arise during construction and design of earth hydroengineering complexes, including those such as estimating the bearing capacity of an inhomogeneous base, determining the lateral pressure of the soil, determining the criterion of crack formation in the cores of dams, estimating the stability of the slopes of structures and so on, cannot be resolved confidently when conducting ordinary model experiments with full-scale soils. The main reason for this is that the stresses due to the natural weight in the models are considerably less than those in real structures: at the same time, as shown by investigations of many authors, the deformation and strength properties of soils under small and large stresses may differ considerably and not only quantitatively but qualitatively as well.

To overcome this difficulty permits the use of the centrifuge modelling method in investigations, the main advantage of which is the possibility of investigating relatively small models made of full-scale soil under stresses similar to full-scale in absolute value and by the nature of distribution. Yet another important advantage of the method is that it permits acceleration of the filtration and consolidation processes Π^2 times (where Π is the modelling scale) when conducting tests with water-saturated argillaceous soils.

Centrifugal modelling studies are being conducted in the Scientific Research Department of Gidroproyekt on a centrifugal device having the following specifications: maximum acceleration of 320 g, effective rotational radius of 2.5 m and maximum model weight of 250 kg. To determine measurements during the experiment, the unit is equipped with strain gages, soil pressure sensors, water level meters and other monitoring-measuring apparatus. Rapidly occurring processes and processes of model breakdown are observed by using a television set.

A large number of different investigations with respect to different hydro-engineering objects was carried out at NIS of Gidroproyekt by using the centrifuge modelling method. The results of some of these investigations are outlined below.

The problem of defining the deformative characteristics of soils and specifically the strain modulus has recently become especially acute with regard to improvement of methods of calculating earth structures and bases. According to the existing design practice, the strain moduli of soils making up earth structures or bases are determined in most cases by the results of compression tests conducted on odometers or stabilizing meters according to standard methods. At the same time many authors have proved that these determinations are far from faultless and that the strain moduli, according to the data of these tests, require additional refinements [2].

Attempts were made in the USSR to introduce correcting coefficients and the results of compression tests. Thus, for example, correcting coefficients were found in [4] on the basis of comparing field stamping tests and tests of the soils of the base on odometers which make it possible to refine the strain moduli from compression tests. However, these coefficients do not take into account the dependence of the strain modulus on stresses. Moreover, the results of field stamping tests themselves with which the comparison was made were inadequately determined and therefore they are used only in designing the foundations of industrial buildings and shallow foundations.

To evaluate the actual deformative properties of the soils of bases and earth structures, attempts were made to determine the strain moduli from data of observations of the settling of completed structures. Thus, it was determined that the mean strain moduli of clays which make up the bases of the concrete dams of the Svirskaya GES [hydroelectric power plant], the Saratovskaya GES and the Volzhskaya GES imeni V. I. Lenin comprise 730, 700 and 1,300 kgf/cm^2, respectively. At the same time the strain moduli of these soils did not exceed 300 kgf/cm^2 in all cases according to data of compression tests.

It should be noted that similar results were also found from observations of the deformations of an embanked structure -- the rock-filled Infernillo Dam (Mexico). The strain moduli calculated by settling of individual layers of the fill exceeded those determined on stabilizing meters more than two times.

The examples given above confirm that in most cases compression tests provide exaggerated characteristics of soil compressibility. One may assume that the main reasons for this are a nonuniform stress field in the sample, stress concentration under the die and the occurrence of plastic deformation zones in the soil as a result of this.

One can avoid these disadvantages by testing samples in a centrifugal force field. This is of very great value with respect to investigating the compression properties of soil since, first, it makes it possible to provide a known stress state in the specimen and to avoid the occurrence of plastic deformation zones under the die and, second, it permits testing large soil samples (weighing up to 200 kg), which makes it possible to obtain statistically more reliable results.

The strain moduli were determined for the upper part of the Nurekskaya rock-filled dam under construction with a core 48 m high at maximum vertical stress of σ = 10 kgf/cm^2. According to this, tests were conducted on soil specimens 240 mm high at centrifugal acceleration of 200 g, which provided creation of stresses in the specimen similar to those in the prototype. The strain moduli were determined under conditions of a uniaxial stress state (Figure 1, a). The specimens were tested in a special cylinder 310 mm in diameter. To reduce the effect of frictional forces, the cylinder walls were made of alternating metal and rubber rings of complex profile which permit the cylinder to be compressed in the axial direction following settling of the soil. The volumetric weight of the cylinder walls was selected as equal to that of the soil, while the maximum axial deformation of the walls somewhat exceeded the possible compression of the specimen.

When testing the specimens in a centrifugal force field, the vertical stress distribution in them were similar to full scale in height, i.e., it increases in proportion to the depth of the considered layer (Figure 1, b). In order to estimate the soil compressibility over a sufficiently narrow range of loads, settling of the layers in height were measured in the specimen by using deep marks (GM), the rods of which in protective pipes were brought out to the surface, and also by using surface marks (PM).

The experiments were conducted on specimens of natural soil of the core of the dam. This soil (sandy loam) had the following characteristics: boundary of flowability -- 19 percent, flattening boundary -- 15 percent and plasticity number -- 4 percent. The specimen was prepared by layer packing of the soil in the cylinder at optimum moisture content (W = 10 percent). The density of the soil skeleton was given within the range of γ_{sk} = 2.03-2.07 g/cm^3, which corresponded to the density of material laid in the dam core. The density and moisture content of the soil were checked by sampling during preparation of the model and also during analysis of it after the experiment.

The tests were conducted by the step stress scheme by increasing the centrifugal acceleration sequentially to 50, 100, 150 and 200 g. The specimen model was held at each stage of loading until total damping of settling, which was recorded by remote strain gages. The length of each experiment was 6-8 hours.

As a result of step loading and also due to the fact that settling of the specimen was recorded at different levels, a significant amount of information was obtained in each experiment which was sufficient to plot the curve

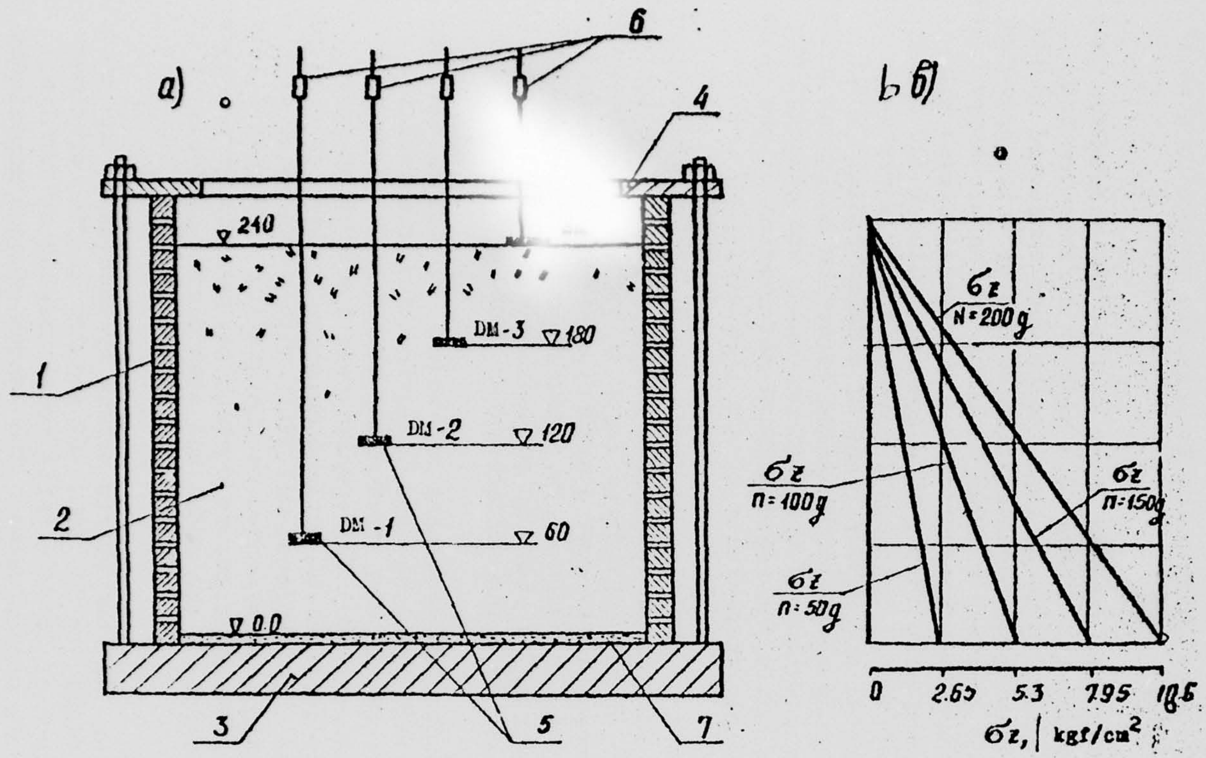

Figure 1. Compression Tests: a -- test cylinder: 1 -- specimen; 2 -- compressible cylinder; 3 -- support plate; 4 -- compression plate; 5 -- deep marks (GM); 6 -- strain gages; 7 -- sand filter; b -- stress distribution in the specimen at different stages of centrifugal stress

of variation of the soil strain modulus as a function of load. The results of testing a specimen having initial composition modulus of γ_{sk} = = 2.07 g/cm^3 are presented as an example in Figure 2, a. One may note that all the values of settling obtained in the experiment are plotted rather well on the approximating curve. This is yet another confirmation of the correctness of the test method used. The curve of variation of the strain modulus found in testing samples of this soil on an odometer is presented for comparison in Figure 2, b. As can be seen, the strain moduli differ essentially two times over the entire range of loads. Thus, the soil strain modulus under a load of 5 kgf/cm^2, according to centrifuge test data, comprises 420 kgf/cm^2, while that according to odometer test data comprises 210 kgf/cm^2.

A cycle of tests to evaluate the danger of crack formation in the core was also carried out for the Nurekskaya Dam.

Analysis of the results of full-scale observations made on many stone-earth dams with cores showed that cracks usually form in the upper zones of the cores near steep shore slopes or at points of a sharp break in the relief of the base. Nonuniform settling and horizontal strains, which are of maximum importance near the crest, cause the appearance of tensile stresses in the soil. If the value of these stresses exceeds the maximum for the given soil, brittle failure occurs in the extended zone and cracks form.

Thus, to know in which case this type of failure may occur, one must know the extent and location of the maximum tensile stresses within the dam with one or another shape of the canyon and the extent of the longitudinal extensibility of the soil. The experiments were conducted on models of a core created in a trapezoidal symmetrical canyon geometrically similar to Nurek Canyon. Models were made from full-scale simulated soil of the core and were installed with given moisture content equal to 11 percent and density of 2.05-2.07 t/m^3. The models were tested during the first stage under plane deformation conditions and the effect of lateral prisms was not taken into account.

The core models were placed in centrifuge carriages and acceleration of them was begun. In this case, to approximate real conditions, the increase of stresses in the models was provided according to a previously compiled work production schedule. This gradual increase of stresses was accomplished by selecting the corresponding centrifuge acceleration mode. The structural settling by the end of each loading stage was assumed overexaggerated and to do this the loading stages were held until practical stabilization of settling. Taking into account that the construction period was simulated only by stresses in the given experiments, the instruments recorded the total structural and operational settling of the core.

It should be noted that the most complete representation of the presence of the assumed process of crack formation in the experiments may be provided by measuring the horizontal movements of the crown. However, taking into

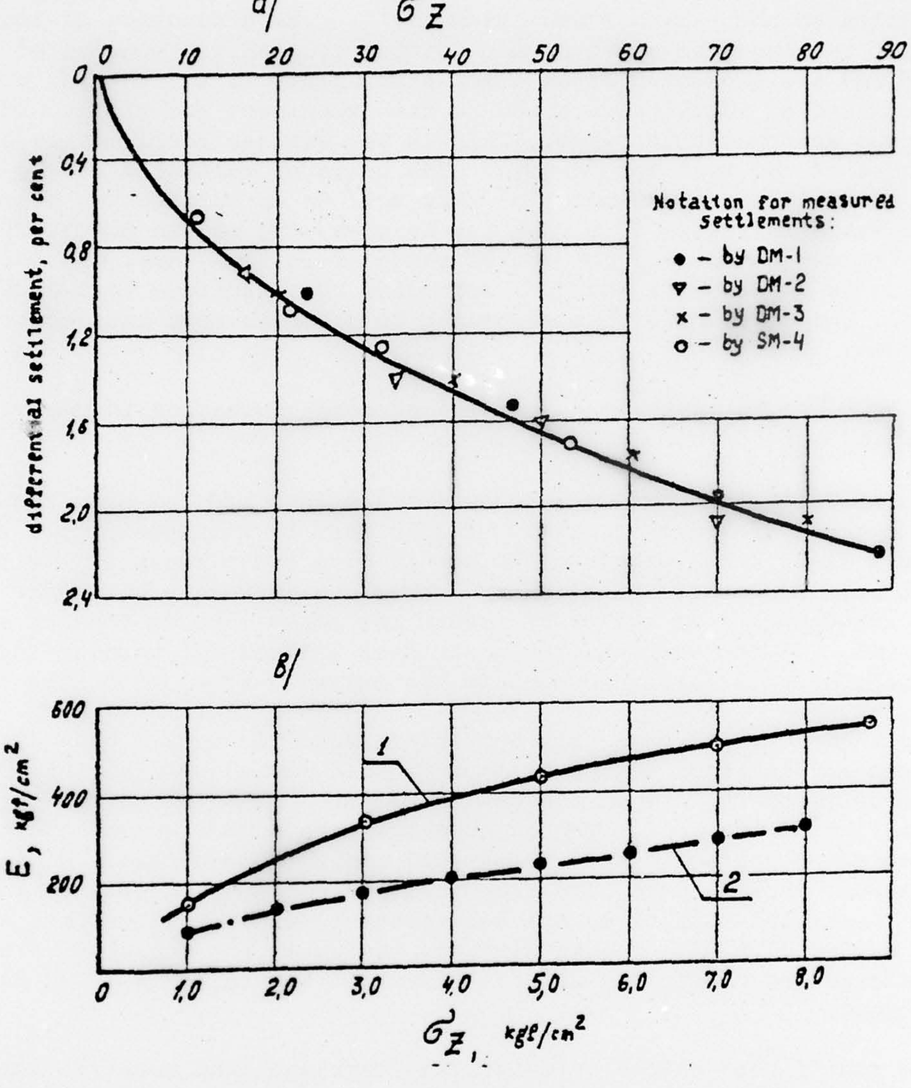

Figure 2. Test Results: a -- relative settling of soil; b -- dependence of strain modulus on stresses: 1 -- centrifuge tests; 2 -- odometer tests

account that only settling is usually observed under real conditions in dams, the criterion of crack formation determined on model should be related to settling for use under full-scale conditions. Nonuniform settling, determined as the first derivative of the settling curve above the core, was used as this criterion. This criterion was used in analyzing the results of the experimental data.

The tests showed that tensile zones form in the upper zone of the dam core near the shore slopes, while a compression zone forms in the central part of the core (Figure 3). It was determined that the location of the tensile and compression zones on the crown of the core does not depend on the degree of nonuniform settling, i.e., the location of the tensile and compression zones remains fixed with proportional variation of the degree of settling and it varies only as the nature of the settling diagram along the crown varies.

It was established that cracks form in the core material of the Nurekskaya Dam (melkozem of pebbly loam) with an excess of nonuniform settling equal to 0.012, which corresponds to maximum extension of the soil of $7.0 \cdot x\ 10^{-5}$. The location of cracks is symmetrical with respect to the canyon axis and their location is strictly confined to the maximum tensile strains. The cracks are easily visible after testing the models and removing the side walls, they are located at a distance of 0.2-0.3 B (where B is the canyon width) and have a depth of 45-50 mm, which comprises 9-10 m when recalculated to full-scale conditions.

The investigations made it possible to determine the maximum extensibility of soil under conditions of a real stress-strain state. The maximum extensibility was previously determined either under uniaxial tensile (figure eight) or pure bending (small beam) conditions [Leonardas and Nahrain]. Moreover, the experimental investigations made it possible to determine at what nonuniform settling of the core there should appear the danger of cracks and consequently in which cases one must be prepared to eliminate them.

Centrifuge modelling is very effective when investigating the stability of the slopes of earth structures (dams, dikes, ditches and so on). This method is essentially the only one which can be used to check one or another method of calculating the stability of a slope or of evaluating the stability of the slope of a real structure.

The main task of the investigations was to determine the nature of deformation and subsequent failure of slopes of different height consisting of adhesive and nonadhesive soils. At the same time the problem of how deformation of the slopes of structures of different types (using the slope of a dam and of an embankment as an example) differ was investigated.

All the investigations were conducted with respect to homogeneous slopes located on a rigid base.

A sand-clay mixture of the sandy loam type was used as the adhesive soil. It was packed in the model with an average moisture content of 16 percent and with density of $\gamma_{sk} = 1.70\text{-}1.72\ t/m^3$. Its strength parameters were determined on a shear device of Gidroproyekt and comprised $tg\,\varphi = 0.706$ C = $= 0.217\ kg/cm^2$.

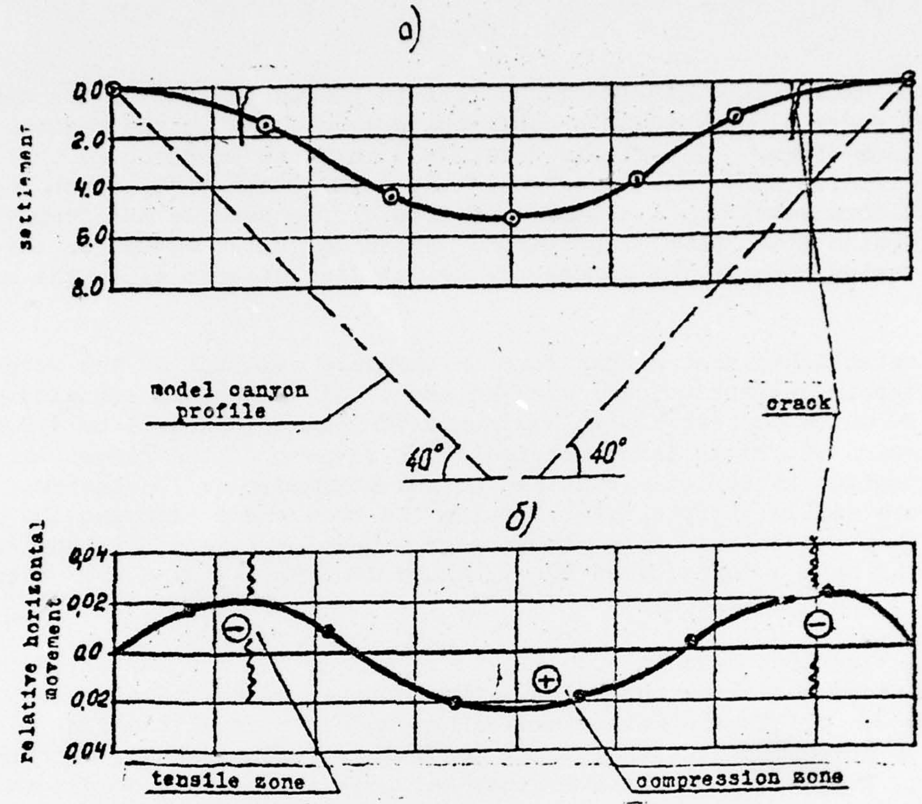

Figure 3. Investigating Crack Formation Conditions in the Core of a Dam (Longitudinal Section): a -- surface settling of core; b -- increase of longitudinal strains

The experiments were conducted in special sectional holders with smooth transparent walls. The inner dimensions of the holders made it possible to investigate models with a slope up to 40 cm high, which corresponds to the height of a prototype equal to 128 m at maximum modelling scale.

The tests were conducted by step loading to evaluate the sequence and nature of deformation of the slope models. That is, stresses similar to those in the prototype of a slope initially 6 m high, then 12, 24, 33 and 60 m high and finally 75 m high, were created sequentially by using centrifugal accelerations in the model. The location of the reference grid was recorded and the surface deformations were measured after each loading. The total deformations of the models were measured by using strain gages and residual strains were measured by using a reference grid. The centrifuge acceleration mode was selected from stage to stage so that maximum consolidation of the specimen in a given stress state occurred during the acceleration.

The entire experiment usually lasted approximately 18 hours together with stops for recording the position of the reference grid.

Experiments with models of adhesive soil were conducted mainly on slopes with an embedding angle of 45 and 90°. The stages of loading the slope corresponded to a prototype height of 12, 24, 36 and 75 m. The results of step testing of a model of adhesive soil with embedding angle of 45° are presented as an example in Figure 4.

The nature of the resulting deformations in the slope during loading or an increase of height are clearly followed by the position of the set of lines of the equal horizontal movement (and settling) or isostaths.

The isostaths of horizontal movements have the shape of circles, each with its own radius, in the model of a slope on a rigid base.

If the height of the slope is increased, the isostaths are shifted toward the base but variation of the shifts occurs out of proportion to the increase of the slope height. The nature of this variation may be written by a function of the dependence of the radius of the isostath of displacement on the height of the slope (Figure 5). The following fact is also interesting: if the height of the slope and consequently the displacement of the isostaths along the profile of the model are increased, the centers of their approximate circumferences are shifted along the same vertical located at a distance of 2-3H from the edge of the slope (H is the height of the slope).

Cracks form on the crown of slopes of models when specific stresses are achieved in all experiments. Although the depth of these cracks was approximately 0.15H, nevertheless their effect on stress distribution is very significant and they must be taken into account when calculating stability. Attempts to take into accounting hardening cracks by introducing the corresponding coefficients were also made previously, but they were approximate in nature and have found little use in design and calculations.

The experiments to determine the critical height of a stable slope permitted evaluation of the role of cracks. Experiments on models of the vertical slopes of the ditch were used for this and the models were tested to destruction. The slope was deformed in the following manner. Cracks formed on the crown when the stresses in the model were increased, these cracks opened up during the experiment and one-two outside cracks began to increase in depth until collapse of the slope occurred along one of them. The typical collapse of the vertical slope found in the experiment is shown in Figure 6, a.

Let us determine the equilibrium conditions for this case. The calculating scheme used is shown in Figure 6, b. The initial data for the calculation, determined by the laboratory method, are as follows: the volumetric weight of the soil skeleton is 1.71 g/cm^3, moisture content is W = 16 percent, the angle of internal friction tg φ = 0.725 and adhesion is 2.60 t/m^2.

The equilibrium conditions of the collapsed side may be written: (with regard to cracks on the crown)

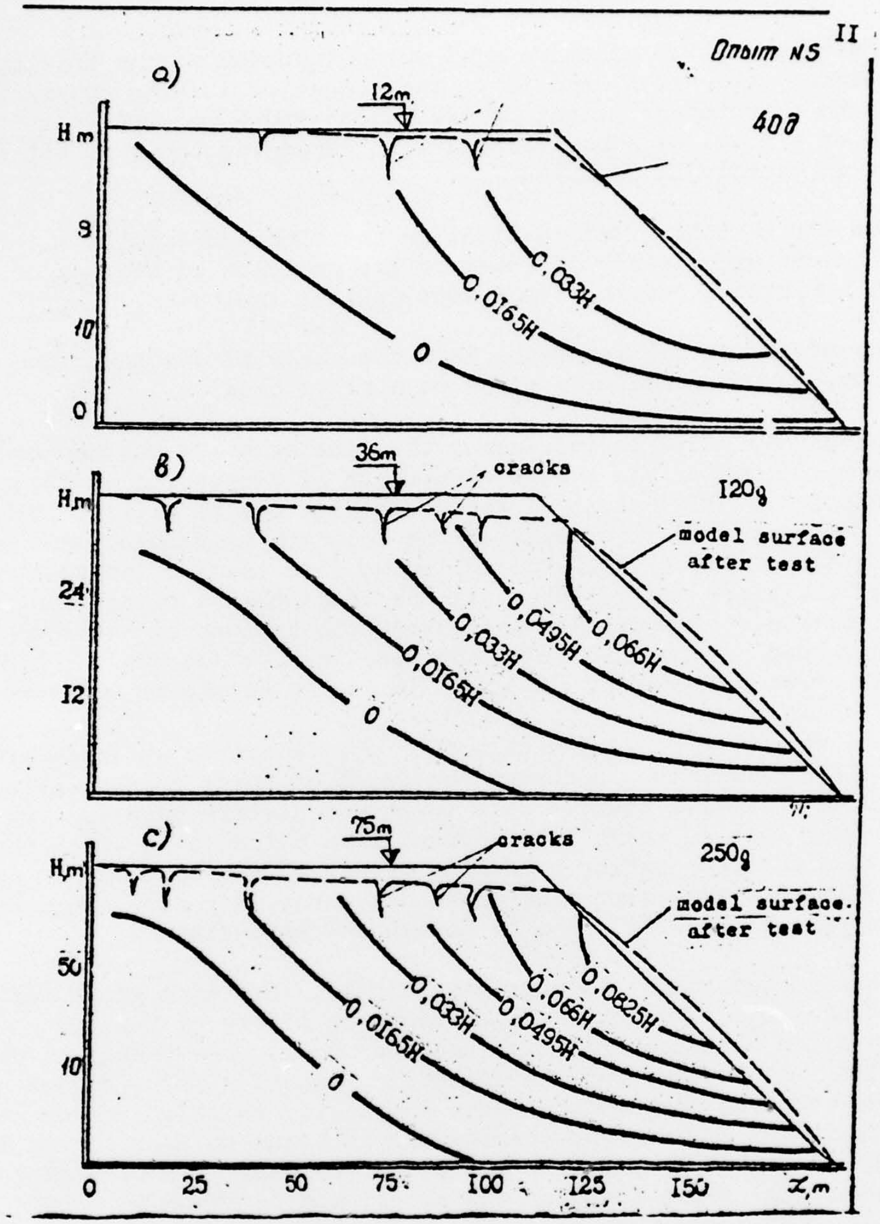

Figure 4. Isostaths of Horizontal Movements During Step Loading of Models

$$P\sin\theta = \frac{c h_{cp}}{\sin\theta} + P\cos\theta \, tg\,\varphi$$

$$\sin\theta\left[\frac{\gamma h^2_{cp}}{2tg\theta} + \frac{\gamma h_{cp} h_r}{tg\theta}\right] - \frac{c h_{cp}}{\sin\theta} - \left[\frac{\gamma h^2_{cp}}{2tg\theta} + \frac{\gamma h_{cp} h_r}{tg\theta}\right]\cos\theta \, tg\,\varphi = 0 \qquad (1)$$

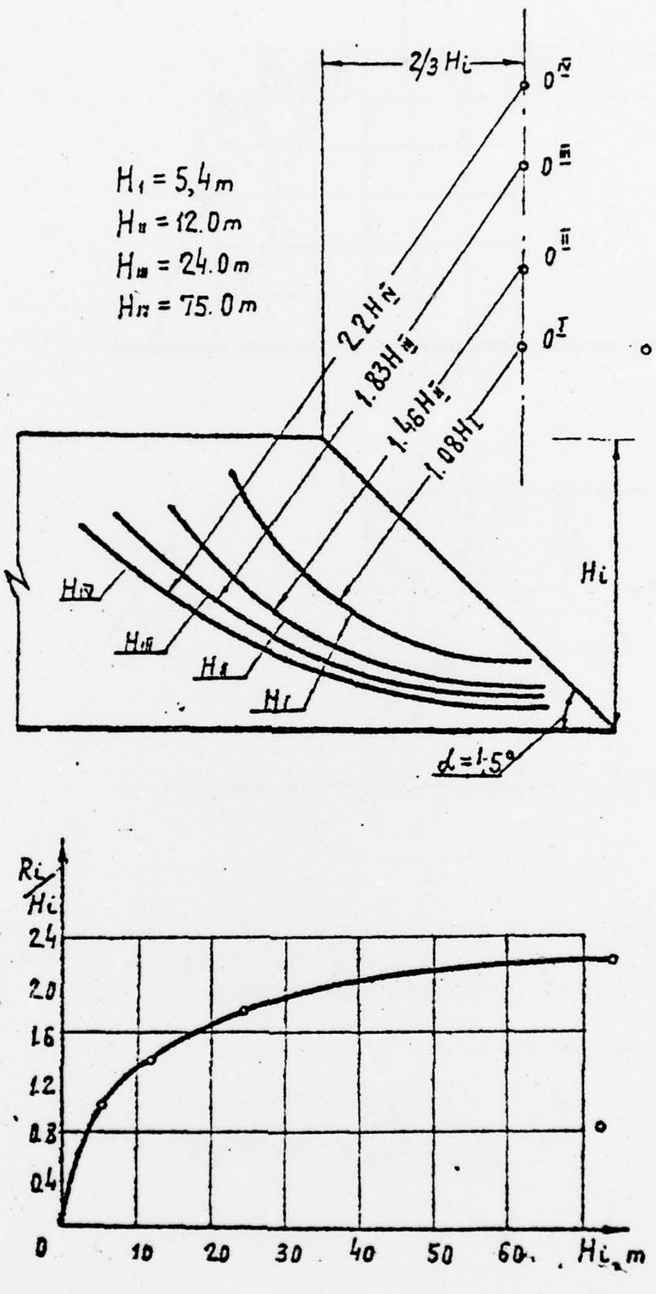

Figure 5. Variation of the Position of the Isostath (0.016H): horizontal displacements with an increase of slope height

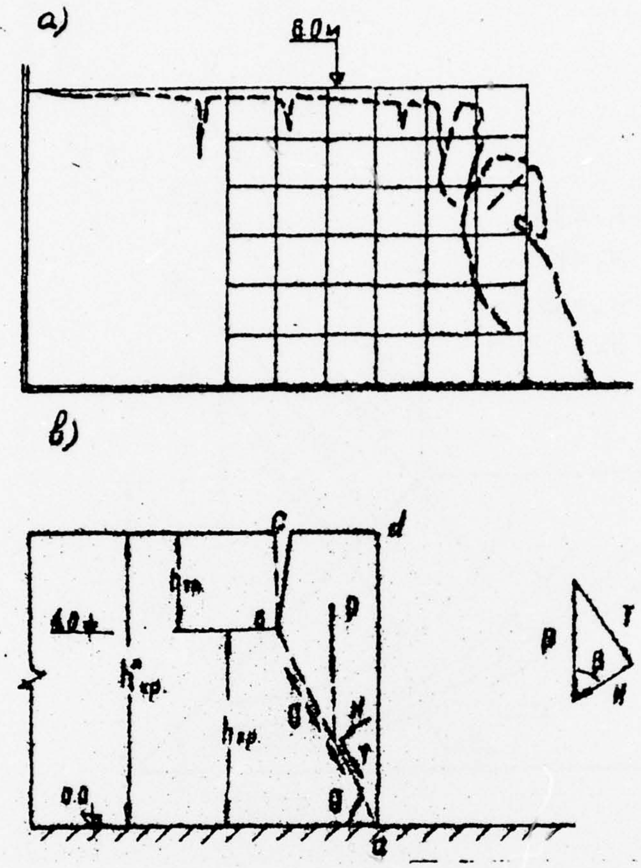

Figure 6. Evaluating the Conditions of Failure of a Vertical Slope: a -- failure after centrifuge testing; b -- calculated diagram of collapse

after transformation we find:

$$h_{kp} = \frac{2c}{\gamma(\cos\theta \sin\theta - \cos^2\theta \, tg\varphi)} - 2h_r \qquad (2)$$

The maximum value of h_{kp} will occur provided that

$$\frac{d}{d\theta}(\cos\theta \sin\theta - \cos^2\theta \, tg\varphi) = 0$$

having solved this equation, we find

$$\theta_{kp} = 45^\circ + \varphi/2$$

hence,

$$\cos\theta \sin\theta - \cos^2\theta \, tg\,\varphi = \frac{1}{2\,tg\,\theta_{kp}} = \frac{1}{2\,tg(45° + \varphi/2)}$$

having substituted into expression (2), we finally find:

$$h_{kp} = \frac{4c\,tg(45° + \varphi/2)}{\gamma} - 2h_T \qquad (3)$$

where 2Ctg(45° + φ/2) is the compressive strength of the specimen in a uniaxial stress state.

Let us consider how accurately the derived expression describes the experimental results. In the experiment the slope collapsed at acceleration of 20 g, which corresponds to the height of the vertical slope of the prototype of 6.0 m and in this case the depth of the hardening crack was 2.0 m (H/3).

Analytical calculation for the soil parameters above yields the following:

$$h_{kp} = \frac{4c\,tg(45° + \varphi/2)}{\gamma} - 2h_T =$$

Thus, the difference of the derived values is very small.

The methods of calculating the stability of slopes existing at the present time do not take into account the type of structure, part of which is the slope. Thus, there is no difference in the method of calculating the stability of the slope of a dam or a natural slope. At the same time full-scale observations of the collapse of different slopes showed that the nature of collapse of a natural slope and the slopes of dams is different. This is determined by the difference in the stress state diagrams in different types of structures.

Taking into account that the method of calculating the stress state of structures and the method of calculating the stability of slopes with regard to real stresses have still not been adequately developed, a cycle of comparative investigations was carried out on a centrifuge to evaluate the nature of deformation of the slopes of structures.

Slopes with an embedding angle of 45°, made of adhesive material, were investigated. Two types of models were tested: a symmetrical dam and a fragment of a natural slope. The models were made from the same soil packed with identical moisture content and density. The tests were conducted simultaneously in two centrifuge holders.

The experiments were conducted by the method of step loading of the model described above. This test scheme made it possible to determine the differences in the nature of deformation of the models at different heights of the simulated prototype.

Let us consider the nature of deformation of a model of a natural slope during loading by its own forces. As the height of the slope, simulated by increasing centrifugal accelerations, increases, a hardening crack 0.15H deep occurs on the crown of the slope at a distance of approximately 0.5H from the edge. With a further increase of stresses, this crack opens up and a number of other cracks appears simultaneously at a distance of 0.25-1.4H from the edge of the slope. There are no cracks on the slope itself, which indicates that the tensile stresses form only along the crown of the slope. Also remarkable is the fact that the cracks essentially do not increase in depth during an increase of stresses, but open up only in width, i.e., the cracks open up to a specific level of extensive compression of the soil corresponding to 0.3-0.4 t/m^2. An increase of crack depth above these maximum values was observed directly prior to collapse of the slope and thus may judge indirectly the degree of loss of stability by the structure by the depth of the cracks.

The deformations of the dam slopes in the lower part up to a height of approximately 0.5H are similar to those of the slope of a ditch. Thus is obvious from Figure 7, where the isostaths of different horizontal displacements of soil in models of the slopes of a dam and ditch are shown. The nature of slope deformation is completely different in the upper part of the slope. Thus, whereas there are no tensile stresses on the surface of the ditch slope, they do occur in the upper zone of the dam slope and cracks on the slope are observed. Moreover, the edge of the ditch slopes undergoes considerable horizontal displacements, whereas the displacements of the crown of the dam are very insignificant. The surface displacements of the models of dam slopes and of a natural slope after the experiment are shown in Figure 7. A difference is also observed in settling, although settling of the dam is greater, but nonuniform settling is less; therefore, the cracks inside the natural slope are considerably wider and their number is greater with equal settling and equal horizontal displacements.

The experiments thus indicate that the method of calculating natural slopes should be distinguished from calculating the stability of dam slopes. As a result of hardening cracks appearing on the crown, a natural slope has less stability than the slope of a dam, all things being equal.

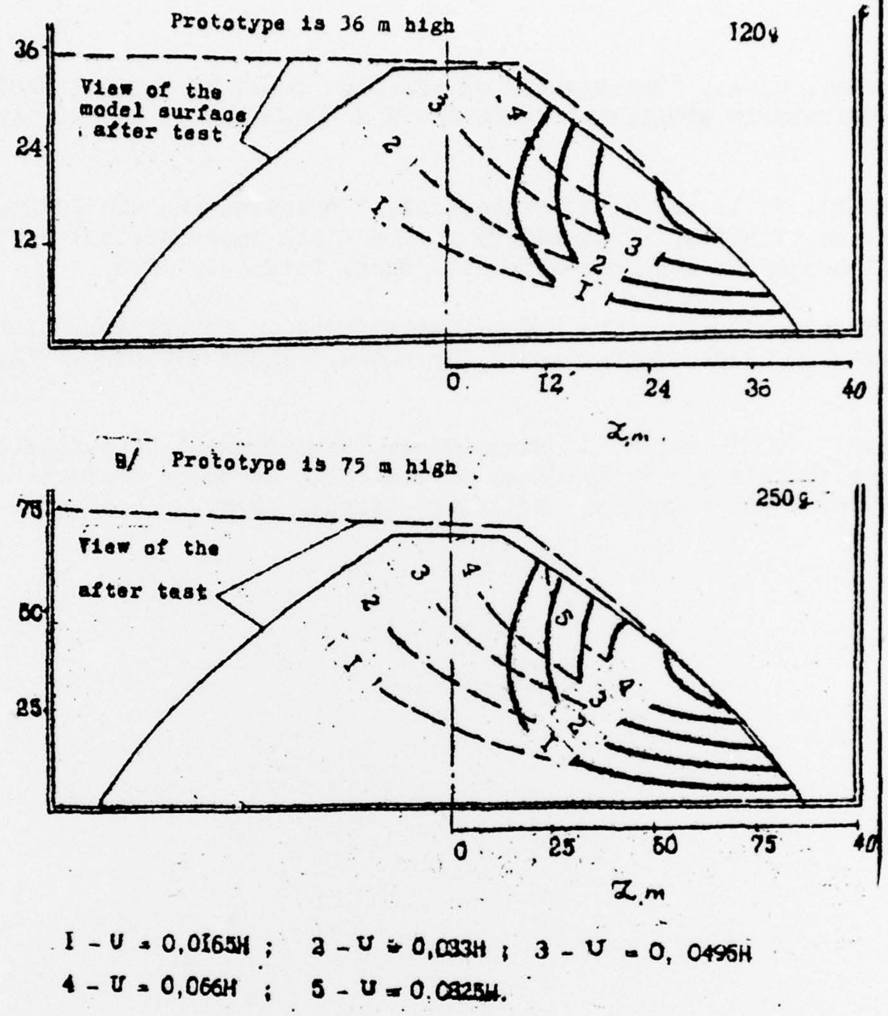

Figure 7. Comparison of Deformations of a Dam Slope and a Ditch From Centrifuge Test Data: ---- -- isostaths of slope horizontal displacements; ——— -- isostaths of dam horizontal displacements

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