SPECIALTY SESSION 4

SPECIALTY SESSION No.4

SOFT SOIL BASES OF CONCRETE HYDROTECHNICAL STRUCTURES

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SOFT SOIL BASES OF CONCRETE HYDROTECHNICAL STRUCTURES. Bengt B Broms (Stockholm, Sweden)

General

The following three topics have been proposed by the Organizing Committee for discussion at Specialty Session 4 on "Soft Soil Bases of Concrete Hydrotechnical Structures" namely:

(1) Foundation strength and stability of concrete hydrotechnical structures on soft soil bases (research and design methods including time effects).

(2) Settlement and horizontal displacements of concrete hydrotechnical structures.

(3) Methods to increase the sliding resistance and bearing capacity of concrete hydrotechnical structures constructed on soft soils (anchorages, drainage, grouting, etc).

The proposed topics are very broad and cover many aspects of the design and the construction of gravity and buttress dams, offshore structures, locks, drydocks, and other concrete hydrotechnical structures. It is characteristic that such structures often are large and costly and that the consequences of failure frequently are disastreous and connected with the loss of human lives. Hydrotechnical structures are in addition generally subjected to high lateral earth or water pressures. Seepage and high uplift pressures often affect their stability.

In this report the loads acting on hydrotechnical structures, the resistance against sliding, bearing capacity, settlements and control methods are discussed.

Concrete Dams

Concrete gravity and buttress dams are normally founded on rock. In the case these structures are constructed on soil the base area is increased to lower the foundation pressure and special provisions are taken to allow movements between adjacent blocks and to prevent failure by piping. The requirements placed on dam sites are constantly increasing while the quality of the sites which are still available is decreasing.

The highest concrete dam which has been constructed so far (1973) is the Grande Dixence gravity dam in Switzerland with a total height of 284 m (Desmeules, 1963). It is followed by the Inguri arch dam in USSR with a total height of 272 m. Both dams are founded on rock. Examples of gravity dams which have been constructed on soft soils are the Lower Svir dam, the Ivankovo dam, the Uglich dam, the Tsymlliansk dam and the Rybinsk dam in USSR (Rousseaux et al., 1961).

The number of gravity and buttress dams being built is decreasing partly because of the high costs of these dam types in comparison with arch and earth dams. It has been estimated that the costs for an arch dam, where it can be used, are between 40 and 70 percent of those of gravity dams (Hupner et al., 1955). The relative costs are dependent, however, on the local conditions and how accessible the site is. It is interesting to note in this connection that in USA more than 90 percent of all dams recently built are earth and rock fill dams while in Switzerland almost 90 percent are concrete dams (Mermel, 1970).

Failures of concrete dams are generally caused by foundation defects rather than by failure of the dam itself. A study of the cause of failure of 40 concrete and masonary dams has indicated that almost half of these have failed due to excessive settlements, by piping or by sliding (Sundquist, 1955).

In the design of gravity or buttress dams the bearing capacity, sliding resistance, settlements and construction difficulties must be considered. Particularly the uplift pressures are critical because of their decisive influence on the sliding resistance. The design of a number of gravity and buttress dams in Romania has been described by Priscu and Diacon (1971).

Gravity dams have the advantage over buttress dams that the contact pressures are relatively low and that the spillway vibrations are less than for buttress dams (Goodhue, 1961). Gravity dams are, however, relatively sensitive to overloading as pointed out by Coyne (1955). Buttress dams can tolerate relatively large differential settlements (Hammond, 1955). They can be adopted to almost any foundation material ranging from sand and gravel to good rock. Buttress dams with a sloping upstream face have the additional advantage that they take advahtage of the water pressure due to the slope of the face.

The construction of a dam does not only interfere with the natural conditions at the site but also with the conditions within the whole reservoir area. This fact had to be considered in a soil exploration program. The foundation investigation required for large dams has been discussed for example by Garnier and Comes (1970).

Geophysical exploration methods are used extensively during the explorating phase to determine such factors as the depth to bedrock and the layer sequence. A special problem is the occurance of boulders or large stones which causes difficulties during the construction of cut-off walls. The required investigation depth is 1.5 to 2.0 times the height of a dam (Hvorslev, 1949). Borings should penetrate soft and permeable materials. They should extend to such depths that seepage settlements and bearing capacity can be estimated with reasonable accuracy.

Offshore Structures

Heavy reinforced concrete structure will be used extensively in the near future in the North Sea as production platforms and for the storage for oil. Such structures have already been used for lighthouses. It is anticipated that these structures will be placed directly on the sea bottom in water depths up to 150 m without previous preparation of the sea bed. They will be subjected to high wave and wind forces which may cause failure of the structures by sliding. The bearing capacity of the underlying soil must be high to resist high eccentric and inclined loads. Also the high uplift pressures which act on the bottom of these structure must be considered as well as local scour and piping around the structures. In addition the gradual softening of clay had to be taken into account when the long term bearing capacity and the long term sliding resistance are evaluated. The liquefaction of uniform fine sand caused by the wave forces should also be considered. Test results indicate that liquefaction can occur even in medium to dense sand if the shear stress ratio is sufficiently high. The difficulties which were encountered during the construction of two reinforced concrete lighthouses have recently been described by Antonakis (1972) and Turner (1965).

The exploration of the soil conditions in water and the retrieval of high quality soil samples are difficult because of the often large water depths and the adverse weather conditions. Static penetrometers as well as different types of drop samplers are commonly used in offshore work for the exploration of the soil conditions close to the sea bottom

Docks and Locks

The size of drydocks has increased considerably during the last few years. For example a drydock for tankers up to 300,000 tons was completed in 1971 (Millars and Hassani, 1971). Drydocks with up to 500,000 tons capacity are under construction.

In most cases dry docks and locks are founded directly on the underlying soil or rock. Piles are only used when the soil conditions are unfavourable.

Dry docks and locks are as a rule subjected to high uplift pressures as well as high lateral earth and water pressures on the dock or lock walls. The high uplift pressures can be resisted by a heavy bottom slab, by piles or by anchors. It is also possible to relieve the uplift pressures through drainage. Design and construction problems in connection with two dry docks on soft soils has been described by Hausser et al. (1964). Excessive settlements are seldom a problem since the weight of the excavated soil for a dock or a lock normally is larger than the weight of the structure (Little, 1948). The soil reaction on the bottom slab is frequently analyzed on the basis that it is supported on a serie of elastic springs (Winkler foundation).

Loading Conditions

In the analysis of the foundation strength and of the stability of concrete hydrotechnical structures the loading conditions must first be considered.

The loads caused by the weight of a concrete structure and by static water pressures can generally be calculated with a high degree of accuracy. Minor variations in the unit weight of concrete can, however, occur since it is affected by the percentage of reinforcement, by the unit weight of the aggregate and by the degree of saturation.

Considerable uncertainty is connected with the evaluation of the wave forces acting on offshore structures. It is particularly difficult to estimated the maximum wave height and the wave period. It is important that the calculations are reliable since the wave forces have a decisive influence on the stability and safety of offshore structures.

Silt deposited in a dam reservoir can cause high lateral pressures on a dam structure. There is some controvercy about the magnitude of these pressures. It seems reasonable, however, that a dam should be designed for a lateral earth pressure equal to that at rest which corresponds to an earth pressure coefficient of 0.5 to 0.6 with respect to the effective pressures in the soil. A silt blanket will reduce the seepage and uplift pressures acting below a dam if the permeability of the silt is less than that of the foundation material. This reduction of the uplift pressure is generally not considered in the design.

The maximum ice pressures on concrete dams is dependent on the thermal expansion of the ice, the thickness and the strength-deformation properties of the ice and on the drag forces from the wind. Ice pressures of up to 300 kN/m can develop along the crest of a dam. For offshore structures in the Artic drifting ice had to considered.

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Earthquakes can have an important effect on the stability of concrete dams, locks and dry docks due to the accelerations transmitted to these structures and the risk of liquefaction in fine saturated uniform sand. Concrete dams and other hydrotechnical structures are commonly designed for a horizontal acceleration of 0.05 to 0.15 of that of gravity. The vertical accelerations are generally neglected. Seed and Martin (1966) have, however, pointed out that the seismic force during a major earthquake can be substantially greater than that corresponding to a seismic coefficient of 0.05 to 0.15. Also the increase of the water pressures caused by earthquakes must be considered in the design.

There are no precise analytical method available for the evaluation of the uplift pressure acting on concrete dams or on other hydrotechnical structures as pointed out by Hammond (1955) and by Yoshida (1958). The effectiveness of grout curtains, sheet pile walls and other cut-off walls and of drainage wells in reducing the uplift pressures is to a considerable extent influenced by minor variations of the geological conditions which are difficult to evaluate even from a relatively extensive soil exploration program.

Drainage is particularly important for the releave high uplift pressures acting under gravity dams, locks and dry docks especially in the case when the soil is stratified or erratic. A reduction of the uplift pressures generally results in a substantial saving of the weight required for stability of a structure with respect to lateral sliding. It has been estimated that a 3 % reduction of the net head corresponds to a 1 % reduction of the concrete volume for a gravity dam.

In the preliminary design of a gravity dam it is frequently assumed that the uplift pressure at the line of drains is equal to one third of the difference in head between upstream and downstream face. The stability against sliding is normally also checked for the case the drains are not functioning and the uplift pressure is uniformly distributed below the base. The U.S. Corps of Enginners assumes in the design of gravity dams with cut-off walls and drainage wells that the uplift pressure varies between 50 and 100 % of the full uplift pressure (Goodhue, 1961). Usually a value of 66.6 % is used. It is furthermore assumed that the uplift acts on 100 % of the base area.

Extensive measurements of seepage and uplift pressures on concrete spillways and gravity dams constructed on alluvium in USSR has been presented by Russo (1958) and by Ginsbourg (1958). An example of measured uplift pressure along the base of a gravity dams is shown in Fig. 1. The very low uplift pressures which were measured indicate that the grout curtain at the upstream face and the drains are functioning properly.

Foundation Strength and Stability

Failure of a soil is normally assumed to occur when the yield or shear strength of the soil has been exceeded.

The yield or shear strength (s_f) is generally expressed in terms of a cohesion (c) and an angle of internal friction (ϕ)

$$s_f = c' + \sigma'_f \; tg \, \phi' \qquad (1)$$

where σ'_f is the effective normal stress acting on the failure plane.

The effective shear strength parameters c′ and ϕ' are normally evaluated from field and laboratory tests. It is common to determine the shear strength in the field with large size direct shear tests. Such tests, however, are primarily used in rock. In some cases in situ direct shear tests have also been used in hard and in stiff fissured clays. In situ direct shear tests have been described by Goodhue (1961), Evdokimov et al. (1970), Toma et al. (1970), Kazimierz (1970), Clarke et al. (1970), Wallace et al. (1970), Paes de Barros (1970) and others. With such tests it is possible to determine both the peak and the residual shear strength. Typical test results are shown in Fig. 2. Many soils and rocks have a peak strength which is considerably higher than the residual strength (Fig. 2 a). The difference decreases in general with increasing normal pressure. A typical stress-strain relationship for soils and rocks which previously have undergone large movements and contain old slip or failure surfaces is shown in Fig. 2 b. In this case the deformations increase with increasing shear stress without reaching a maximum value.

The shear strength of cohesive soils (clays and silts) is normally determined by drained or consolidated-drained triaxial tests with pore pressure measurements. Large diameter samples are preferred (10 to 15 cm). Small diameter (3.7 to 5.0 cm) samples are commonly used. When the sample size is too small the cohesion of the soil can be overestimated especially if the soil is fissured. The residual strength is frequently determined by laboratory direct and ring shear tests.

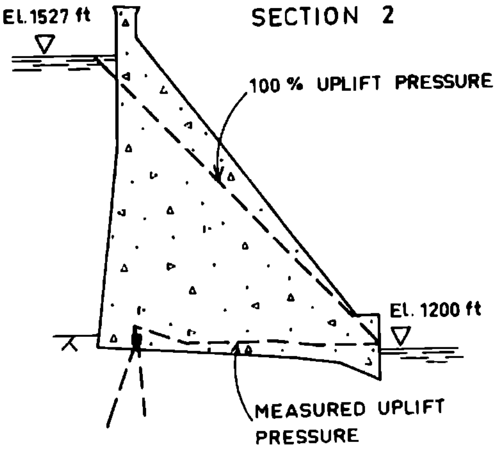

Fig. 1. Measured uplift pressures at the base of Detroit dam, Oregon, USA. (After Bloor, 1955)

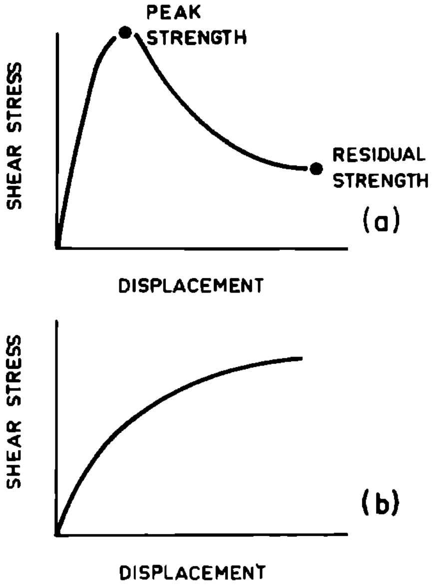

Fig. 2. Stress-strain relationships

The angle of internal friction of sand and gravel is generally determined by drained triaxial or direct shear tests. Stage loading is sometimes used. In some cases the angle of internal friction determined from triaxial tests is higher than that from direct shear tests. In most cases the difference is small. The friction angle can for special purposes also be evaluated from plain strain tests. The angle of internal friction by this method is often 3 to 4 degrees higher than that from triaxial tests when the relative density of the sand is high. When the relative density is low the difference is generally small (Cornforth, 1961).

In the case the soil conditions are erratic it is preferable to investigate a relatively large number of samples so that the results can be analyzed statistically. The shear strength values which are selected for the analysis of the sliding resistance are often lower limiting values. Statistical methods are seldom used in the evaluation of the test results. Such a procedure will result in an unknown safety factor as pointed out by Peck (1965).

Failure of a gravity concrete structure occurs when the sliding resistance of the soil along the base of the structure or the bearing capacity of the soil at the toe of structure has been exceeded.

Sliding Resistance

Examples of failures which at least partly have been caused by sliding are that of the Austin Dam in Texas in 1900 which was constructed on limestone, clay and shale and that of the St. Francis dam in California in 1928 which was built on schist and conglomerate.

The resistance against sliding is normally analyzed by the Coulomb-Mohr failure criteria (Eq. 1). The corresponding factor of safety (F_s) is calculated from the equation

$$F_s = \frac{tg\,\phi' \Sigma N + c' A}{\Sigma H} \qquad (2)$$

where ΣN and ΣH are the total vertical and total horizontal force acting on the structure, respectively, and A is the area of the horizontal plane considered.

The U.S. Corps and Engineers have for several gravity dams required a minimum factor of safety of 1.9 with respect to sliding parallel with bedding planes (Bloor, 1955). The calculated factor of safety for the spillway dam at the Lenin hydroelectric station on the Volga is 1.58 under normal conditions and 1.14 under extreme conditions (Borovoi et al., 1965). The corresponding calculated factor of safety with respect to sliding for the spillway dam at the Saratov hydroelectric station on the Volga is 1.8.

The inclination of bedding planes will have a large influence on the sliding resistance. Failure by sliding is especially critical in soils and rocks which contain old failure or slip planes particularly when slip planes are horizontal or dip in the direction of the inclined load.
An other factor which is important with respect to failure by sliding is the gradual softening which take place in some soils and rocks during the construction when the overburden is removed. It is not uncommon that the shear strength of stiff fissured clays decreases from 1.0 MN/m^2 to between 0.2 and 0.03 MN/m^2 or more due to infiltration of water as pointed out by Terzaghi (1936). It is thus necessary to protect many stiff or hard clays, shales and soft sandstones against such deterioration with bituminum or to leave some of the soil or rock in place until just before the pouring of the concrete.

The risk of progressive failure is large in stratified and fissured soils. Failure initiates at points close to the upstream or the downstream face of the dam where the shear stresses are the highest and progresses towards the center of the dam.

In stiff fissured clays a reduced shear strength between the peak and the residual shear strength is frequently used in the analysis of the sliding resistance. This reduced shear strength can be expressed in terms of a residual factor R defined by the expression

$$R = \frac{s_f - s}{s_f - s_r} \qquad (3)$$

where s_f and s_r are the peak and residual shear strengths, respectively and s is the average shear strength at failure (Skempton, 1964). An analysis of slope failures indicates that large variations of the residual factors can exist. The residual strength represents a lower limit. Values between 1.0 and 0.75 are commonly used in the design.

It appears reasonable that the factor of safety under extreme loading conditions should be at least 1.1 à 1.2 with respect to failure by sliding when a shear strength which corresponds to the residual strength of the soil is used in the analysis.

The residual cohesion (c'_r) is usually equal to zero or very small. The residual angle of shear resistance (ϕ'_r) is frequently 1 to 2 degrees less than the angle of internal friction (ϕ') which corresponds to the peak strength of the soil. In some cases the difference can be as large as 10 degrees (Skempton, 1964). In preliminary designs a value of $tg\,\phi'_r = 0.3$ is frequently used. In the design of the spillway for the Mangla Dam which is founded on sandstone and fissured overconsolidated clay a value of $tg\,\phi'_r = 0.3$ was chosen (Sherlock et al., 1970). In the case the soil or rock contains montmorillonite seams or layers the residual angle of shear resistance can be very small.

In fissured soils the shear strength as determined by conventional tests does not necessarily bear any relation to the values which governs the stability as pointed out by Skempton (1964). For example the size of the soil sample has for such soils a large influence on the measured shear strength. In the case an old slip surface or plane exists in the soil the shear strength along the slip plane will correspond to the residual strength even when there has been no renewal of movement for several thousand years. Experience indicate that failures in such soils are preceeded by relative large movements which accelerate before failure.

The mechanism of progressive failure has been attributed by Skempton (1964) and by Skempton and LaRochelle (1965) to stress concentrations at fissures. Bjerrum (1967) has on the other hand attributed progressive failure to stress concentrations at the toe of a slope. The relative danger to progressive failure is considered by Bjerrum to be high for unweathered and weathered overconsolidated plastic clays with weak bonds. For overconsolidated clays with low plasticity the relative danger of progressive failure is very low.

Earthquakes, blasting or sudden changes of the pore water pressure can cause liquefaction in fine uniform sand. This effect must be considered in the design of hydrotechnical structures. The factors which contribute to liquefaction have been investigated among others by Seed and Lee (1966), Seed (1968) and by Finn et al. (1970).

Liquefaction is caused by the gradual buildup in pore water pressure which takes place in saturated sand under cyclic loading when drainage is prevented. The soil looses suddenly its strength when the pore pressures approach the total overburden pressure and the effective stress in the soil drops to zero. The development of liquefaction is dependent of the shear stress ratio τ / σ'_v where τ is the maximum shear stress and σ_v the initial effective overburden pressure. Other factors which contribute are the number of load cycles, the initial relative density and the drainage conditions. Also the previous stress history of the soil is important as shown by Finn et al. (1970). Test data indicate that the number of load cycles required to induce liquefaction increases very rapidly with increasing initial relative density. Seed (1968) has pointed out that the risk of liquefaction is larger for level ground than for a slope.

In many cases liquefaction will only occur locally, especially when the sand is medium dense to dense, since liquefaction only develops within a certain relatively small deformation range. Liquefaction of a thin sand seam confined between two clay layers will probably initiate close to the edge or periferi of a structure and spread laterally towards the centre due to the uneven shear stress distribution below a rigid structure. The movements will likely stop as soon as the motions from the earthquake stop.

The liquefaction potential of a sand cannot be determined reliably from cyclic load tests on prepared samples. Undisturbed samples are required as pointed out by Finn et al.(1970).

An analysis by Seed (1968) indicates that flow slides caused by liquefaction have occurred at earthquakes with 5.3 to 8.8 in intensity and with epicentres located a few miles to hundreds of miles from the location of the slide. The soil has usually been loose to medium dense with a standard penetration resistance less than 20 blows per ft. An investigation of the failure of the Sheffield dam in California in 1925 indicated that the factor of safety with respect to sliding along the base could have been as low as 0.75 due to liquefaction of the loose silty sand near the base. The pore pressure increase during the earthquake was evaluated by cyclic undrained direct shear tests. The liquefaction potential of a soil can probably be evaluated more accurately by cyclic direct shear tests than by cyclic triaxial tests.

The liquefaction potential is decreased if the soil previously has been subjected to cyclic loading because of interlocking of the soil particles. The cyclic loading from a small earthquake or by a small storm will thus decrease the risk of liquefaction below gravity or buttress dams or below offshore structures. Previous liquefaction and large shear deformations will on the other hand increase the liquefaction potential as indicated by Finn et al.(1970).

The risk of liquefaction can be decreased by drainage of trapped sand layers or lenses, by compaction or by an increase of the weight of the structure.

For dams founded on rock with a height exceeding 100 m the so-called shear friction factor of safety (F_f) is used to evaluate the resistance against sliding. This safety factor is defined by the expression

$$F_f = \frac{f\,\Sigma V + rsA}{\Sigma H} \qquad (4)$$

where f is a friction factor, r is the ratio of the average and the maximum shear stress, s is the shear strength of the foundation material, ΣV and ΣH are the total vertical and total horizontal force, respectively and A is the base area of the dam. The value of 0.5 is normally taken for the coefficient r and values between 0.65 and 0.75 for the friction factor f. A minimum value of F_f of 4.0 is generally required (Hammond, 1955, Bloor, 1955). However, the derivation of Eq. (4) and the basic assumption made are not clear.

Many values have been suggested for the friction factor f and for the sliding coefficient between concrete and the underlying soil or rock as discussed for example by the French National Committe on Large Dams (1961). However the bond between concrete and the underlaying foundation material is normally sufficient to force the failure through the soil or the rock.

Friction coefficients of 0.65 to 0.75 are frequently used which correspond to a friction angle of 33 to 37 degrees. A sliding factor as high as 0.845 was used in the design of the Gierra dam in Scotland (Roberts, 1955). In India values up to 0.8 are common (Iyengar et al., 1955). For the Miranda buttress dam in Portugal values as low as 0.64 to 0.66 were used in the analysis (Henriques, 1961). For the Inga dam in Zaire a sliding factor of 0.7 was selected (Afschrift and Pahud, 1961). The U.S. Corps of Engineers requires that the ratio of the all horizontal and vertical forces including uplift should be less than 0.65. When earthquake forces are considered this ratio can be as high as 0.85 (Bloor, 1955).

The base width to height ratio is about 0.75 for gravity and buttress dams of moderate height founded on rock and about 0.80 or larger for high dams. If uplift is not considered a base width to height ratio of about 0.65 should be adequate (Chapman and Campbell, 1955).

Bearing Capacity

The soil at the toe of a gravity or buttress dam is subjected to loads which are both eccentric and inclined. Both factors have an appreciable influence on the ultimate bearing capacity. Under normal loading conditions the resulting force should be located within the middle half of the base. Under exceptional loading conditions a location of the resultant within the middle half may be acceptable.

The ultimate bearing capacity q_{ult} of a soil is usually evaluated from the well known equation

$$q_{ult} = I_c K_c c N_c + I_\gamma K_\gamma N_\gamma \gamma g B + I_q K_q D \gamma g N_q \qquad (5)$$

where I_c, I_γ and I_q are factors which reflect the influence of the inclination of the applied load, K_c, K_γ and K_q are shape factors; N_c, N_γ and N_q are bearing capacity factors which are functions of the angle of internal friction ϕ, B is the width of the loaded area, D is the depth below the ground surface and γ is the unit weight of the soil. General shear failure with well defined failure or slip surface has been assumed in the analysis. The slip surfaces are generally assumed to be circular or spiralshaped.

In compressible soils the deformations can be so large that they will be well beyond the limit where the structure ceases to behave as intended in the design. In saturated cohesive soils a deformation of 3 to 7 percent of the foundation width is frequently required to reach failure. In cohesionless soils the deformation to reach failure corresponds to about 5 to 15 percent of the foundation width (Vesić, 1973). The deformation required to reach failure increases with increasing foundation depth.

For a relatively long and narrow structure ($K_\gamma = 0.5$) founded on sand or gravel ($c = 0$) close to the surface ($D = 0$) Eq. (5) is simplified to

$$q_{ult} = 0.5\, I_\gamma N_\gamma \gamma g B \qquad (6)$$

The bearing capacity factor N_γ in this equation can be evaluated numerically. Large variations in the values on N_γ has been obtained by different researchers. The angle of internal friction used in the evaluation of the bearing capacity factor N_γ is dependent on the average normal stress along the failure surface as pointed out by deBeer (1965). Test data indicate that the mobilized angle of internal friction decreases with increasing size of the foundation. This decrease is probably caused by progressive failure in compressible soils and by the curvature of the failure envelope. Also zones of weakness in the soil affects the ultimate bearing capacity. Vesić (1969) indicates that very large foundations may fail by punching at a stress which is much less than that calculated by Eq. (6). The reduction of the bearing capacity has by Vesić been expressed in terms of a compressibility factor. Test results as shown in Fig. 3 (DEGEBO) indicate (Muhs and Kahl, 1954; Kahl and Muhs, 1957; Muhs, 1961) that the measured ultimate bearing capacity can be larger than that calculated when the relative density of the soil is low and smaller than calculated when the relative density is high. It should be pointed out that test results are only available for relatively small plates or slabs and that considerable uncertainty is connected with the extrapolation of the test results to large structures.

The ultimate bearing capacity decreases very rapidly with increasing inclination of the applied load as illustrated for a cohesionsless soil in Fig. 4. It should be observed that failure at low normal loads occurs by sliding along the base and that the failure becomes more deepseated with increasing normal load. The decrease of the ultimate bearing capacity with increasing inclination is partly compensated by a decrease of the average normal pressure along the failure surface and by a corresponding increase of the angle of internal friction due to the curvature of the failure envelop curve.

Seepage pressures at the toe of a gravity or buttress dam or around the perimeter of an offshore structure will have a large influence on the ultimate bearing capacity. A vertical upward seepage gradient of (i) has the same effect as a reduction of the submerged unit weight of the soil by $i\gamma_w$ where γ_w is the unit weight at water.

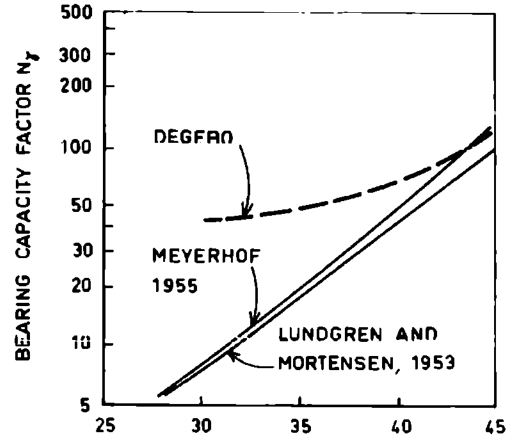

Fig. 3. Bearing capacity factor, N_γ (after deBeer, 1965)

An other factor which had to be considered in the evaluation of the bearing capacity is failure by piping due to internal erosion especially in silty soils. This risk can be decreased by inversed filters, by drainage or relief wells at the toe of a dam and by cut-off walls as discussed for example by Terzaghi (1948). Also the erosion at the toe of a dam can appreciably decrease the bearing capacity.

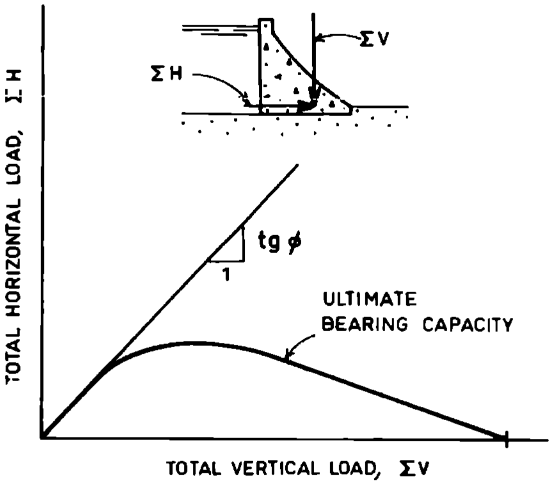

Fig. 4. Ultimate bearing capacity

Settlements and Lateral Displacements

It is important that the settlements or the lateral displacements of concrete hydrotechnical structures should not be so large that the deformations impair the proper functioning of the structures. Relatively little is, however, known about the total and the differential settlements that locks, dry docks, gravity or buttress dams can tolerate. Frequently relatively large total settlements can be permitted while the allowable differential settlements are small due to the high stiffness of reinforced concrete structures.

The settlements of structures on clay are normally calculated from Terzaghi's classical consolidation theory with a compressibility index and a coefficient of consolidation determined from oedometer tests. Often only a very small number of samples are tested. This method has been modified by Skempton and Bjerrum (1957). The stress-strain relationships determined by triaxial tests have also been used in settlement calculations as is the case in the stress-path method proposed by Lambe (1964).

Plate load tests are common in sandy soils as pointed out, for example, by Bjerrum (1963). The results from dynamic and static penetrometer tests can also be used for settlement calculations in sand (deBeer 1965, Schmertmann, 1970, Meyerhof, 1965). Penetration tests have the advantage that a relative large number of points can be investigated and that variations of the soil properties in the vertical and the horizontal directions can be determined. In silt and overconsolidated clay the Menard pressiometer can be useful.

Sherbina and Dundukoff (1958) has compared calculated and measured settlements of several dams and power stations constructed on soft soils. The authors found that the calculated settlements from oedometer tests were two to three times larger than the measured values.

The finite element method has been used to evaluate the settlements and the lateral deformations of such structures as dams, dry docks, locks, retaining walls, bracing systems and bulk heads. With this method it is possible to take into account such factors as the construction sequence and complicated boundry conditions pointed out for example by Desai (1972). The finite element method requires access to high speed computers with large storage capacity.

Many soil-structure interaction problems are three dimensional. Finite element methods exist which take three dimensional effects into account. These methods are, however, complicated and costly. Most three dimensional problems are therefore treated as plain strain problems.

One of the main difficulties with the application of the finite element method in soils engineering is the complex multiphase nature of the soil and the evaluation of the soil properties. Such factors as residual stresses and the presence of joints and fissures complicates the problem. However almost all finite element programs treat soils and rock as a continuous medium without cracks and fissures. Approximate solutions are available which consider joints and fissures. In most cases very simple idealized stress-strain relationships are used.

In many cases it is also important to consider the construction sequence since dewatering, excavation, pouring of concrete, placement of back fill and the restoring of ground water table can have important effects. In locks and dry docks the displacement of the walls and of the base slab as well as the downdrag by the fill placed around the structures influence the behaviour. The downdrag is mainly dependent on the rigidity of the foundation and of the back-fill material. Clough and Duncan (1969) have analyzed the behaviour of two U-frame locks by the finite element method. A comparison between calculated and measured deformations for the Port Allen lock is shown in Fig. 5. It can be seen that the agreement is good between measured and calculated deformations.

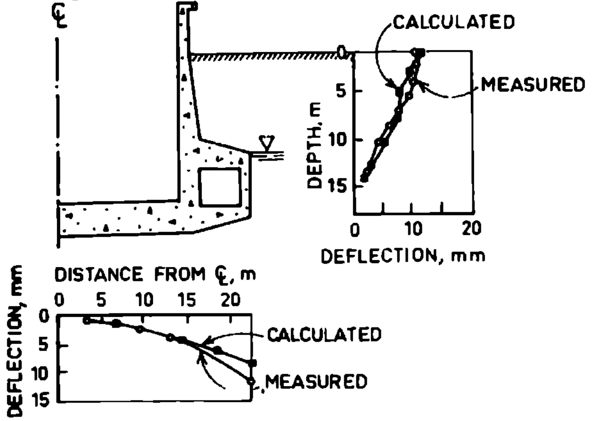

Fig. 5. Predicted and observed deflections for Port Allen lock (after Clough and Duncan, 1969)

In many problems consolidation and creep are important and had to be taken into account. Finite element methods which considers consolidation has been developed by Huang et al. (1971). Creep effects have been included by Nair and Boresi (1970) and by Zienkiewiecz (1971). Methods have been developed by Clough and Duncan (1969) with which it is possible to analyze the effects of excavation and dewatering. Methods have also been developed for overconsolidated clays which take into account the strain-softening of the soil at large deformations.

An example where the finite element method has been used to analyze the deformations below a dam and a powerhouse has been described by Lane (1970). The main purpose of the study was to investigate the effects of settlements on the misalignment and the distorsion of the machine axis.

In Fig. 6 is shown the pressure distribution below the spillway for the Mangla Dam (Sherlock et al., 1970). The pressure distribution has been calculated by a conventional method and by the finite element method. It can be seen that in this case there are appreciable differences betwcen the two methods. It should however be observed that the pressure distribution calculated by the finite element method depends to a considerable extent on the assumed value on the Poisson's ratio and on the modulus of elasticity and how these parameters vary with depth.

The accuracy of the finite element method is primarily governed by the accuracy of the soil constants used in the analysis. Futher advancements are primarily governed by the development of improved field and laboratory test methods rather than by improved methods of analysis.

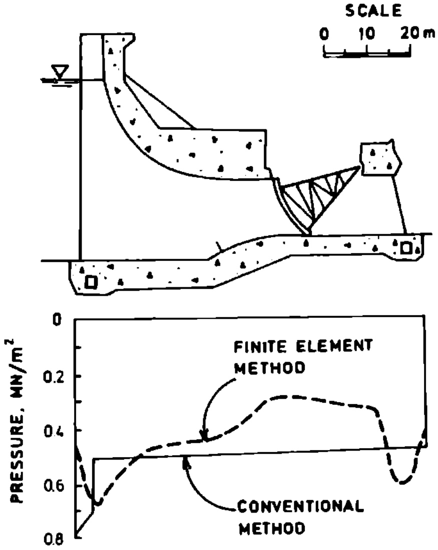

Fig. 6. Foundation pressure distribution (after Sherlock et al., 1970)

Control Methods

It is important to check the behaviour of gravity structures both during the construction and under operaing conditions. By comparing the measured values with those used in the design it is often possible to evaluate the actual safety of a structure. The instrumentation should be such that any changes which can impair the proper functioning of the structure or of its safety are detected in time so that corrective measures can be taken. It is important to instrument not only the structure itself but also the foundation below the structure. Observations should made down to the depth where the soil or rock is not affected by the structure. This critical depth corresponds normally to 1.5 à 2.0 times the width of the structure. The costs of instrumentation, the taking of the readings and the evaluation of the results correspond normally to 1 to 2 percent of the total costs of a structure.

The type of measurements and the accuracy of the readings are dependent on the type of structure, the geotechnical and the geological conditions at the site and on the consequences of a failure. It is necessary that the observations are carried out quickly, that the evaluation is simple and that the results are immediately useful as pointed out by Huggenberger (1970).

The instruments should be simple and rugged and they should function properly during the entire life of the structure. Simple and rugged instruments are preferred before sophisticated and delicate instruments which will give very accurate results for only a limited time (Wilson, 1967; Wilson and Squier, 1969). Corrosion is often a problem during long term measurements. It is desirable that all instruments can be removed and checked periodically (Muller et al., 1970).

The type of measurements which should carried out in concrete dams was discussed for example at the Third Congress on Large Dams in 1948. A summary of available methods has been presented by Hanna (1973).

Settlements, Horizontal Displacements and Tilt

Measurements of vertical and lateral displacements of gravity structures are essential in the evaluation of the performance of dams (Eidelman and Tchernyatin, 1970), of locks (Kaufman and Sherman, 1964) and of other hydrotechnical structures. The lateral displacement of the dam should be checked particularly during the filling of the reservoir. An incipient failure is characterized by large lateral displacements and by displacement rates which increase with time. Particularly the joint movements are important to monitor.

Several methods have been developed during the last few years with which it is possible to measure lateral displacements at the surface and at depth. Lateral displacements are normally determined with plump bobs or by triangulation. By triangulation the lateral displacements can be determined with an accuracy of up to 0.5 to 1.0 mm. Triangulation is time consuming and dependent on the weather. A trained surveyer is required for the measurements. It is, however, frequently difficult to obtain stable reference points. Movements can occur up to several hundred meters from a dam site. The lateral displacements in galleries can also be measured with invar wires or tapes with an accuracy of up to 0.1 mm.

Plump bobs or pendulums with up to 50 m length are commonly used to determine the lateral displacements and the tilt of a structure. The readings can be made by a technician. If the plumb bob shaft extends into rock reverse or floating plumb bobs are used.

Inclinometers can be used to measure the lateral displacements of a structure. An inclinometer consists in principle of a pendulum. Its inclination is measured at successive levels in a cased hole by strain gages or by other types of transducers attached to the pendulum. The lateral displacement can be determined by repeating the measurements at different times. Several types of inclinometers are in use (Kallstenius and Bergau, 1961, Wilson, 1962).

Different types of settlement gages are available as described by Hanna (1973). Settlements are normally determined by precision leveling. An accuracy of 0.2 mm can be obtained with this method. Vertical movements also can be determined by borehole extensiometers. With this method the movements are measured by wires or rods which are inserted into a borehole. The lower end of the wires or rods is anchored to the sides or to the bottom and the upper end is attached to a dial indicator or to a transducer. Hydraulic settlement gages have been developed by Lauffer and Schober (1964) and by Bergdahl and Broms (1967).

The tilt or the rotation of a structure can be measured with tiltmeters. Sensitive selfrecording tiltmeters are available with which it is possible to measure the tilt with an accuracy of ± 1.2 seconds.

Uplift and Pore Water Pressures

The resistance against sliding is to a large extent dependent on the pore water pressures at or just below the base of concrete gravity or buttress dams, locks, dry docks or offshore structures. It is important to continueously monitor these pressures with pore pressure gages so that changes can be detected.

A pore pressure gages should be accurate and durable. Temperature changes and changes of the barometric pressure frequently affect the readings. Pore pressures in pervious soils such as sand and gravel are normally measured with open stand pipes. Stand pipes cannot be used in soils with medium to low permeability because of the large time lag. In these soils pneumatic, hydraulic or electrical piezometers are used.

Pneumatic type piezometers utilize a sealed porous tip which contains a gas or fluid operated diaphragm or valve connected to one or two lines. When the applied pressure in the line is equal to the fluid pressure at the porous tip the diaphragm or valve opens. Air is used to operate the Warlam piezometer (Warlam and Thomas, 1965) while in the Glötzl piezometer fluid is used (Lauffer and Schober, 1965). Pneumatic piezometers are generally simple to operate. They have a small time lag and are stable.

Hydraulic type piezometers consist of a porous tip which is directly connected with one or two pressure lines. Many different types are commercially available. Single line piezometers have the disadvantage that air, which collects in the line, can affect the readings. Air can be flushed out of the system when double lines are used. Hydraulic type piezometers are generally simple to operate and they are stable.

Electrical piezometers are often unreliable for long term measurements. They are generally very accurate for short term measurements and have a very short time lag. They are suitable for the measurement of rapid changes of pore pressures such as those caused by earthquakes or blasting.

Strengthening of Concrete Hydrotechnical Structures

The most favourable dam sites have already been used in many countries as mentioned previously. When the foundation conditions become less favourable the need to improve the strength and deformation properties of the underlying soil or rock becomes important. Several different methods have been proposed to increase the sliding resistance and the bearing capacity and to limit the deformations of hydrotechnical structures.

The most effective way to increase the sliding resistance of a dam is by decreasing the uplift pressures with impervious blankets or aprons, with cut-off walls or through drainage. A cut-off wall can consist of sheet piles, a slurry trench wall or a grout curtain as indicated in Fig. 7. The effectiveness of cut-off walls have been discussed for example by Casagrande (1961) and by Marsal et al. (1971).

Sheet Pile Walls

Steel sheet pile walls are primarily used in cohesionless soils which do not contain large stones or boulders which prevent the driving of the sheet piles. The effectiveness of a sheet pile wall can be improved with bentonite and cement grout. The grout is injected just in front of the wall in order to seal any openings between the individual sheet piles. Steel sheet pile walls are common in USSR (Borovoi et al., 1965).

Grout curtains

Grout curtains are primarily used in coarse grained soils with a total thickness of up to 200 m. An effective grain size larger than about 1 mm is required to ensure penetration of cement grout. Chemical grouts can be used in soils with a permeability larger than 10^{-3} cm/sec. The spacing and depth of the grout holes and the grouting pressure depend on the geological conditions. The manchet method developed by Solètanch is commonly used to allow grouting at different depths (Cambefort, 1967). There is, however, some question about the permanency of the grout due to the leaching.

Slurry Trench Walls

The use of slurry trench walls in the construction of hydrotechnical structures is increasing as discussed for example by Bienvenu and Ract-Madoux (1970). In this method 2 to 5 m long narrow slots are first excavated. The walls are stabilized during the excavation by bentonite slurry. After excavation to final depth the slots are filled with concrete. Up to 100 m deep cut-off walls have been constructed. Also precast concrete panels have been used as well as intersecting piles.

Aprons

Impervious concrete or clay aprons or blankets placed upstream of a dam are very effective in reducing the uplift pressures. For earth dams up to 1000 m long clay blankets have been used (Marsal et al., 1971). The effectiveness of such impervious blankets have been discussed for example by Lellavsky (1965). An example of a gravity dam which has been provided with an upstream concrete apron is the Svir dam in USSR. The apron was loaded by a thick sand fill to increase the sliding resistance. The uplift pressure below the apron was reduced by a drainage system (VBB, 1965). Other gravity dams which have been provided with upstream concrete aprons have been described by Rousseaux et al (1961) and by Borovoi et al,(1965). The apron used for the Tsymliansk spillway dam in USSR is shown in Fig. 8.

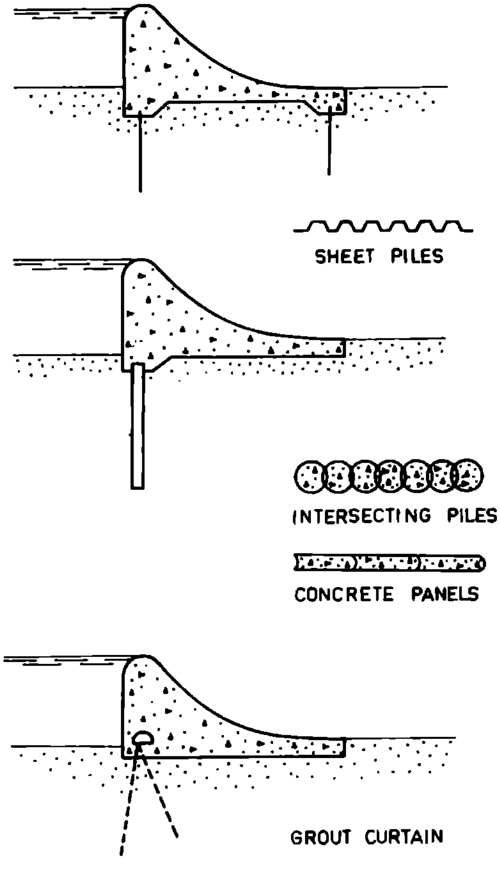

Fig. 7. Cut-off walls

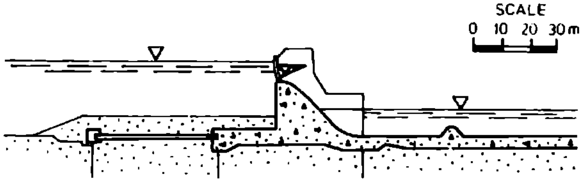

Fig. 8. Tsymliansk spillway dam, USSR (after Rousseaux et al., 1961)

Prestressed cables or rods

The sliding resistance can also be increased by with prestressed cables or rods or with rock bolts (Sundquist, 1955). Rock bolts are commonly used in Sweden for lighthouses which are subjected to high lateral forces from waves and ice. The method has also been applied by the Corps of Engineers (Goodhue, 1961).

Examples where the resistance against sliding of gravity and buttress dams has been increased with prestressed cables have been described by Clarke et al. (1970) and Lancaster-Jones (1969). The sliding resistance has been analyzed using an angle of shearing resistance which was slightly larger than the residual angle of shear resistance of the foundation material. Posttensioned steel cables have also been used to increase the stability of the John Hollis Bankhead dam in Alabama, USA (Thompson, 1969). The load in each anchor was in this case 3.4 MN.

The Cheurfas dam in Algeria was strengthened by steel cables in 1927. This dam, which was constructed in 1882, has a maximum height of 33 m. The cables were inserted in holes with 25 cm diameter which where drilled 4 m apart from the crest of the dam. The same method was used at the heightening of the Steenbras dam in South Africa, the Tansa dam in India and of the Allt-na-Lairige dam in Scotland (Banks, 1955).

Steel cables have also been used to resist the high uplift pressures which act on dry docks. Lackner (1962) has described several cases where this method was applied.

Shear Keys

In some cases it might be advantageous to use shear keys or a stepped base particularly in stratified soils and rocks or to use a sloping base to intersect any weak layers or zones.

Summary

Attention has been focused on this report on the effects of progressive failure, liquefaction and size on the sliding resistance and the stability of concrete hydrotechnical structures and on the use of the finite element method in the analysis of settlements and horizontal displacements. Different methods to strengthen dams, drydocks, locks and offshore structures have also been discussed. It is expected that these problems will become more important as the size of concrete hydrotechnical structures increases and sites with relatively poor foundation conditions are utilized more and more.

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Now I pass the word to Vice-Chairman Mr. Prof.P.D.Evdokimov (USSR)

The difference between the performance of foundations of retaining concrete structures and that of foundations of other industrial and civil engineering edifices lies mainly in the fact that the former are subjected not only to vertical,but also to horizontal loads.The bearing capacity of non-rocky foundations is evaluated proceeding from analytical solutions to problems of the theory of limit equilibrium of granular media based on the assumption of limit equilibrium throughout a non-rocky structure foundation /R. 1,2,3,4/. The ultimate load diagram for the foundation from these solutions has the form of an infinite wedge. Actually, according to experimental findings obtained at the VNIIG, its shape is different, being identical for both ultimate tangential and normal contact stresses. Experimental data demonstrate the effect of structure rigidity on the stress state of the foundation. Thus, in the sand foundation of a rigid footing a wedge is formed under vertical loading; the wedge remaining in the pre-limit state, even though the ultimate bearing capacity of the sand foundation be reached. Hence solutions are being worked out in the USSR to mixed problems. So far solutions have been obtained for cases of vertical loading only /R. 5,6/. Therefore approximate solutions are justified in cases of inclined loading when the resultant of the external ultimate load is sought for given a pre-set contact stress diagram either uniform or trapezoidal /R.7/. Consideration of the dead weight of the soil in plotting the sliding lines makes them dip inward into the depth of the foundation at small modelling numbers. This is corroborated experimentally.

The above results testify to the occurrence of plane shear in the foundation of a rigid retaining structure at small modelling numbers, N_σ, when the sliding surface coincides with the base of the structure.

The necessity to verify the theory, as well as the requirements imposed by the hydraulic construction practice led to an experimental investigation on the bearing capacity of non-rocky foundations of hydraulic structures and further research into contact stresses using models up to 3.5 m wide.

The study was based on similarity criteria of the stress state of non-rocky structure foundations. The data obtained by shearing of plates are presented in Fig.1. The rectilinear section of the graph in Fig.1a corresponding to plane shear lengthens with the foundation width, B. When transformed into the graph of $N_{\tau ult} = f(N_\sigma)$ (see Fig.1b) the graphs of Fig.1a merge into a generalized curve of the bearing capacity of a non-rocky foundation, which proves the validity of similarity criteria. $N_{\sigma cr}$ (Fig.1b) is termed the critical modelling number; plane shear occurs at $N_\sigma \leq N_{\sigma cr}$.

The pattern of deformations in the foundation of a retaining structure at plane shear is depicted in Fig.1c.

In Figs 1d and 1e a partial foundation soil upheaval increasing in function of N_σ is exhibited at the downstream face of the retaining structure at $N_{\sigma p} > N_{\sigma cr}$.

The deformation pattern at $N_\sigma = N_{\sigma\ fail}$ is displayed in Fig.1f. It should be emphasized that footings were sheared on cohesive soils at $\sigma_m = \sigma_p$.

The second similarity criterion $\gamma_m = \gamma_m \frac{B_p}{B_m}$ was not observed, experiments were run at $\gamma_m = \gamma_p$. This goes to increase the margin of safety in evaluating the kind of deformation over the base of plates and prototype structures. In every case plane shear was observed.

According to the USSR Codes and Standards in force at present, the value of the ultimate bearing capacity of non-rocky foundations

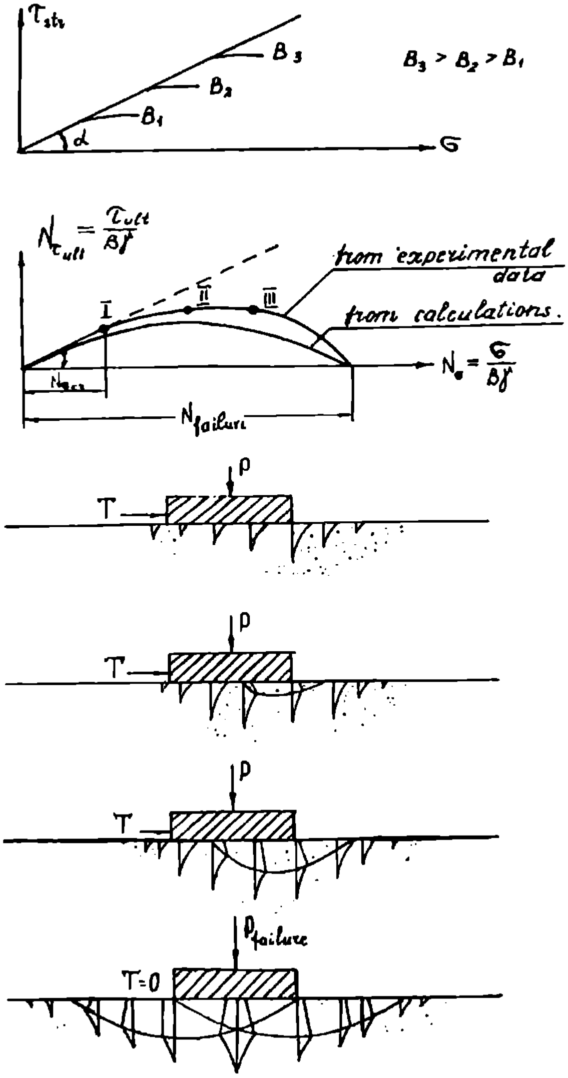

Fig.1

of retaining structures is to be determined with reference to the kind of shear (translational shear, inclined shear) and the type of foundation deformations encountered in practice, viz.

-plane shear, mixed shear
-deep shear.

In the majority of cases up to $N_\sigma \leq 3$ the calculation procedure for plane shear should be used. The analysis carried out revealed that almost all retaining structures have modelling numbers $N_\sigma < 3$. Alongside with the problem of the type of deformation of a non-rocky foundation of a retaining structure, of major importance is the question about its design characteristics of strength. Comparison of the a values along the initial length of $\tau_{ult}=f_\sigma$ and the $tg\alpha$ value (fig.1a) obtained by shearing of plates during experimental studies on foundations of retaining structures to the values of C and φ obtained by laboratory shearing and triaxial compression tests on undisturbed samples revealed a marked discrepancy between them. The values of a and $tg\alpha$ describe the integral strength of the foundation soils to be established by shearing of footings. Hence Codes and Standards envisage shearing tests in the design for major structures (those of the 1st and 2nd class).

Extensive studies conducted into the strength of non-rocky foundations of hydroplant dams erected on the Volga, Dnieper, Dniester, Kama and Daugava proved to be of great benefit. In all the above instances the integral characteristics of foundation strength a and $tg\alpha$ were recommended and accepted as design values being considered sufficiently reliable and, as a rule, higher than the soil characteristics c and φ which used to be adopted by designers based on laboratory test data on soil samples before shearing of footings was introduced into practice. A striking example is furnished by the design and operational experience of the concrete dam of the Gorki hydroelectric plant. In designing the dam the shear coefficient of 0.5 was accepted which is the highest design value so far for clayey soils. The Gorki dam is the lightest and possesses the most efficient profile among the USSR dams, and, possibly, in world practice (R.8).

Settlements of retaining structures are evaluated by summarizing the settlements in each individual layer. The active depth is defined as the soil thickness where normal stresses due to external loading amount to 0.5 γH_a of the stresses induced by the dead weight of the soil.

On the basis of studies conducted at the B.E.Vedeneev VNIIG calculation procedures making an allowance for the increase in the soil deformation modulus across the depth may be considered to give the closest approximation to the settlements recorded at hydraulic structures /R.9/. Since the design and construction of the Nizhne-Svir hydroelectric plant in the nineteen twenties the classical arrangement for enhancing the stability of retaining structures remains an anchored reinforced concrete foreapron. However, the performance of such dams as the Gorki, the Kakhovka, the Volzhskaya appears to be quite satisfactory without aprons (Fig.2). Their stability is increased by an upstream cantilever overhang utilizing the weight of the water above the divice. To prevent partial liquefaction of the sand foundation sheet piling is driven into the river bed.

Experience gained in the USSR in the design and construction of dams resting on non-rocky foundations demonstrates the efficiency of the stability and settlement calculation techniques recommended in Building Codes and Standards.

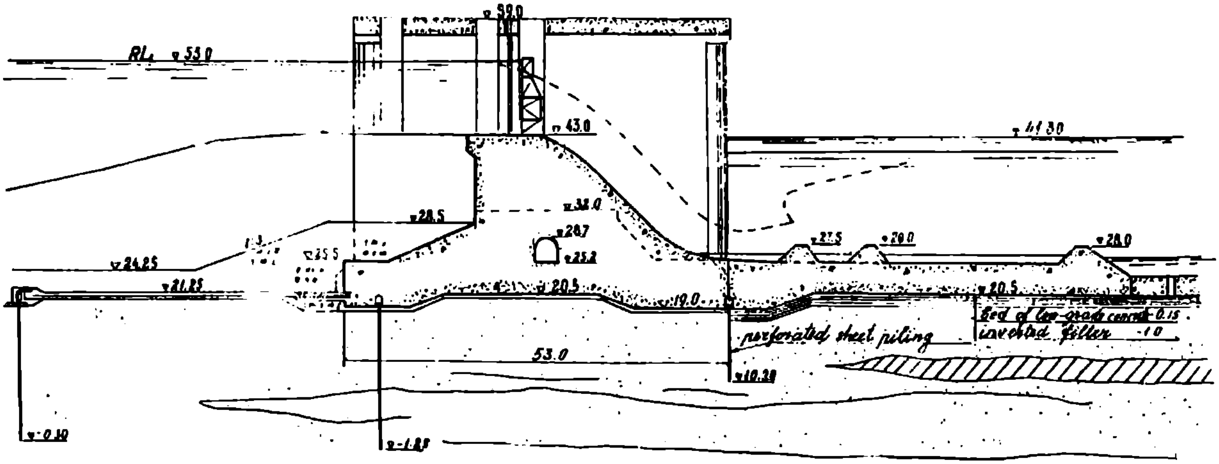

Fig. 2

Chairman Prof. Bengt B.Broms

Thank you Mr.Evdokimov for your interesting contribution.
Mr Isbash, will you please.

J.V.Isbash (USSR)

VARIATIONS OF THE VALUES OF POISSON'S RATIO AND OF THE MODULUS OF THE GROUND STRATUM DEFORMATION UNDER LOCAL LOADING

The scientific basing of the value of the settlement of the ground stratum is connected with a number of problems, the most important being the determination of the modulus of general deformation and the determination of Poisson's ratio. The adequate physical interpretation of them and the determination of a real value are the main criteria for predicting the structure settlement.

To determine the modulus of deformation and Poisson's ratio a volume of subsoil should be chosen below the settlement plate. The cross-section of such subsoil volume is equal to the area of the settlement plate.

Vertical deformations of layers may be meassured by means of depth marks in the limits of the given subsoil volume to the depth 2.0-2.5 diametres of the settlement plate. Horisontal marks are placed in the middle section between each two vertical marks along the lines forming the given subsoil volume. Such placing of marks makes it possible to determine the value of vertical and horizontal relative deformations under any stressed state. The data obtained give the possibility to calculate the value of Poisson's ratio for each singled out volume of the subsoil between the marks.

The analysis of the experimental data shows that Poisson's ratio value constantly changes along the line forming the cylinder or prism. Poisson's ratio value depends on physical properties of the subsoil and the depth of the given section below the toe of the settlement plate. For the medium- granular sand in compact state with the void ratio of 0.52, Poisson's ratio in the initial stage of loading (to 1 kgs/cm^2) at the depth of about 1 diameter of the settlement plate has the value which conciderable exceeds 1. The Poisson's ratio value decreases to 0.5 or 0.4 with the increasing of loading. If the same sand has the void ratio 0.65, the value of the Poisson's ratio changes in process of loading from 1.1 to 0.35 for the depth of measuring equal to the settlement plate diameter. In the same sand but of loose structure (void ratio equals 0.73) the value of the Poisson's ratio in the process of loading varies from 0.80 to 0.32.

Along the forming line of the compressed cylinder (with a round settlement plate) or the compressed prism (with a square settlement plate) the largest values of the Poisson's ratio is at the depth: for a dense sand 1.0-1.2 of the settlement plate diameter; for the sand of a middle density at the depth of 0.7-1.0 of the settlement plate diameter, for a loose sand at the depth of 0.6-0.9 of the settlement plate diameter. The value of the Poisson's ratio falls in the direction of the ground surface and with the increase of the depth.

For the clay subsoil of semihard or hardplastic consistency the Poisson's has the value less than 0.1 under the loads up to 3 kgs/cm^2 only under the pressure of 4-5 kgs/cm^2 it becomes equal to 0.23. For the clay subsoil with the consistency of 0.5-0.8, the Poisson's ratio equals 0.15-0.30.

While measuring the deformations in layers and the mean value of vertical stresses one can determine the value of the general deformation modulus for each subsoil layer in the intervals between the marks.

The analysis of the data obtained indicates that under the similar physical subsoil characteristics the value of the deformation modulus increases with the increase of the subsoil depth.

The above-mentioned data indicate that one should not take for a constant the value of the Poisson's ratio and the deformation modulus for each geological layer within the limits of the compressed thickness.

The subsidence and the inclination of structures calculated without taking into account variations of Poisson's ratio and general deformation modulus might greatly differ from those observed in practice.

Chairman Prof. Bengt B.Broms

Thank you Mr. Isbash for your contribution.
Now I want to invite Mr. Vutsel (USSR)

V.I.Vutsel (USSR)

ON ANALYSIS OF FOUNDATIONS OF HYDRAULIC STRUCTURES BY DEFORMATIONS

The design analysis of hydraulic structures foundations as well as of the prototype settlement and displacement values leads to the conclusion that the necessity has come of basing the foundation design not only on the first limit state (the ultimate bearing capacity), but also on the second limit state (the ultimate strain), taking into consideration the limit equilibrium zones (shear zones h_s)

The following two component parts contribute to the total value of settlement and displacement: the quasi-instantaneous part and the part due to creep stain.

The first part of settlement is determined as the seepage consolidation settlement, and the second part is the secondary consolidation settlement (for clayey soils).

The paper deals with the model of a foundation with the finite thickness and a crack extending below the front face of the structure (fig.1).

<u>The quasi-instantaneous displacement</u> can be approximately determined by the variational Kantorivitch-Vlasov's method developed by I.E.Milejkovsky and N.N.Leontjev, using the differential equation /1/, /2/:

$$\frac{E_0}{1-\mu_0^2} J_{11} \cdot V'' - G_0 \cdot b_n V + q_g = 0;$$

in which $\mathcal{V}$- generalized displacement;
q_g- generalized force expressed in t/m.

For the design layer with the thickness of $H=0.4B+h_s$, (fig.1.) the following displacements can be obtained:

I. Displacement of a concrete structure at $h_s=0$ (single-layer foundation with the thickness H); a) due to the shearing force Q_o:

$$u_1=\frac{Q_o}{E_{oI}\delta\cdot\lambda_o};\ \lambda_o=\frac{E_{oII}}{E_{oI}}\cdot\frac{\sqrt{1-\mu_o}}{2(1-\mu_o^2)}+\frac{B}{2H(1+\mu_o)};$$

b) due to the seepage force q^s_o at $x=0$:

$$u_1^s=\frac{q_g^s\cdot H}{\delta\cdot G_{oI}}\left(1-\frac{E_{oII}}{E_{oI}}\cdot\frac{1}{Sh\,\alpha B^s+\frac{E_{oII}}{E_{oI}}\cdot Ch\,\alpha\cdot B^s}\right);$$

in which:

$$q_g^s=q_o^s\cdot\left(\frac{H}{3}\div\frac{H}{2}\right);\quad q_m^s=\gamma_w\cdot J_m\cdot\delta;$$

$$\alpha=\frac{1{,}225}{H}\sqrt{1-\mu_o};$$

q_o^s- is assumed being uniformly distributed in the zone B^s;
B^s- is determined by the location of the downstream drainage system;
J_m- is the average gradient in the zone B^s; $\delta=1m$.

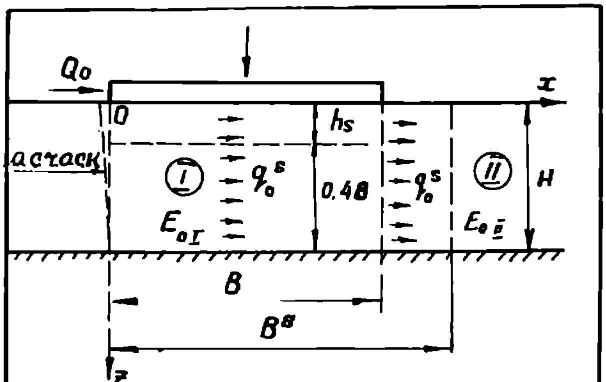

Fig. 1

2. Displacement at h_s equalling the shear zone depth (two-layer foundation, Fig. I):

$$U_2=\frac{h_s\cdot Q_o}{G.B.\delta}\cdot\left(\frac{H-h_s}{h_s}+\frac{1}{I-m}\right);$$

in which m is the plastic deformation degree determined in the load plate tests at the structure foundation; for clayey soils $m\approx 0.6\div0.7$;

G- is the shearing modulus beneath the shear zone;

h_s- is assumed to equal the design depth of the shear zone below the downstream face of the structure.

Creep displacement is recommended to be determined by N.N.Maslov's formula /3/.

Study of prototype observations results (the "Hydroproject" Institut) demonstrates that analysis of foundations of concrete hydraulic structures based on deformations leads to a more economic design of hydraulic structures.

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Chairman Prof. Bengt B.Broms

Thank you Mr.Vutsel for your discussion.

Now Mr. Sobolevsky will you please.

Sobolevsky U.A. (USSR)

The mechanical anisotropy is supposed to be absent while solving the problem concerning the initial stress in anisotropic pervious soils.

Using the following equation the initial stress distribution of anisotropic pervious soils under a quickly applied load may be defined.

$$\begin{cases}\sigma_x+\sigma_y=0;\\[4pt] \dfrac{\partial\sigma_x}{\partial x}+\dfrac{\partial\tau_{xy}}{\partial y}+\gamma_0\dfrac{\partial H}{\partial x}=0;\\[4pt] \dfrac{\partial\sigma_y}{\partial y}+\dfrac{\partial\tau_{xy}}{\partial y}+\gamma_0\dfrac{\partial H}{\partial y}=0;\\[4pt] \dfrac{\partial^2H}{\partial x^2}+\dfrac{K_y}{K_x}\dfrac{\partial^2H}{\partial y^2}=0,\end{cases}\qquad (I)$$

where

$$H=\frac{P_0}{\gamma_0 \pi}\left(\operatorname{arctg}\frac{\sqrt{\frac{K_x}{K_y}}\,y}{x-b}-\operatorname{arctg}\frac{\sqrt{\frac{K_x}{K_y}}\,y}{x+b}\right), \tag{II}$$

K_x - coefficient of permeability in horizontal direction

K_y - coefficient of permeability in vertical direction

The expressions for the stress components f for two cases have been obtained:

a) the horisontal water permeability prevalence over the vertical one (peat, varved clays and etc.)

$$\left.\begin{matrix}\sigma_x\\ \sigma_y\end{matrix}\right\}=\pm\frac{P_0}{\pi}\,\frac{1+\frac{K_x}{K_y}}{1-\frac{K_x}{K_y}}\left(\operatorname{arctg}\frac{y}{x-b}-\right.$$

$$\left.-\operatorname{arctg}\frac{y}{x+b}-\operatorname{arctg}\frac{\sqrt{\frac{K_x}{K_y}}\,y}{x-b}+\operatorname{arctg}\frac{\sqrt{\frac{K_x}{K_y}}\,y}{x+b}\right),$$

$$\tau_{xy}=\frac{P_0}{\pi}\,\frac{1+\frac{K_x}{K_y}}{1-\frac{K_x}{K_y}}\left(\frac{1}{2}\ln\frac{(x-b)^2+y^2}{(x+b)^2+y^2}-\right.$$

$$\left.-\frac{\sqrt{\frac{K_x}{K_y}}}{1+\frac{K_x}{K_y}}\ln\frac{(x-b)^2+\frac{K_x}{K_y}y^2}{(x+b)^2+\frac{K_x}{K_y}y^2}\right), \tag{3}$$

b) the vertical water permability prevalence over the horizontal one (loess soils and etc.)

$$\left.\begin{matrix}\sigma_x\\ \sigma_y\end{matrix}\right\}=\pm\left[\frac{2P_0}{\pi}\,\frac{\sqrt{\frac{K_x}{K_y}}}{1-\frac{K_x}{K_y}}\left(\operatorname{arctg}\frac{y}{x-b}-\operatorname{arctg}\frac{y}{x+b}\right)-\right.$$

$$\left.-\frac{P_0}{\pi}\,\frac{1+\frac{K_x}{K_y}}{1-\frac{K_x}{K_y}}\left(\operatorname{arctg}\frac{\sqrt{\frac{K_x}{K_y}}\,y}{x-b}-\operatorname{arctg}\frac{\sqrt{\frac{K_x}{K_y}}\,y}{x+b}\right)\right],$$

$$\tau_{xy}=\frac{P_0}{\pi}\,\frac{\sqrt{\frac{K_x}{K_y}}}{1-\frac{K_x}{K_y}}\left(\ln\frac{(x-b)^2+y^2}{(x+b)^2+y^2}-\ln\frac{(x-b)^2+\frac{K_x}{K_y}y^2}{(x+b)^2+\frac{K_x}{K_y}y^2}\right) \tag{4}$$

The expressions characterizing the initial stress state of the isotropic medium may be obtained from equation (3) and equation (4) by means of the ultimate transition according to the Lopital rule.

The initial percolation conditions and boundary expressions for the strip width under the load 2b (Fig.1) were used for solving the problem on the initial stability of the quickly loaded soils according to V.V.Sokolovsky method.

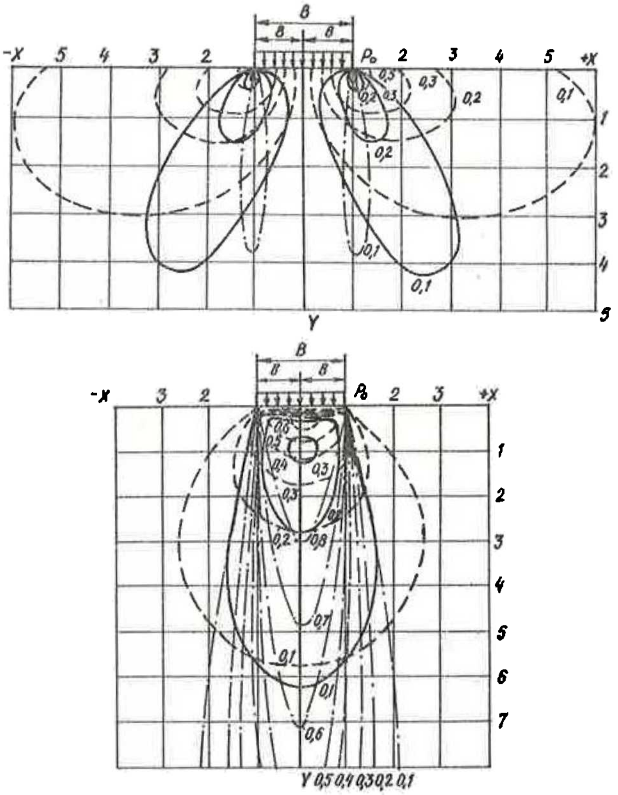


Fig.1. The distribution of equal initial tangential stresses τ_{xy}

a) solid lines-for isotropic soils; dash lines-for anisotropic soils when $\frac{K_x}{K_y}=100$; dot-and-dash lines for anisotropic soils when $\frac{K_x}{K_y}=\frac{1}{100}$

b) the same lines for equal normal stresses σ_y and σ_x

Calculations for finding ultimate loads and slip nets for the saturated anisotropic soils are given in Figure 2.

The calculations for the load $P_o=2C_o$ (where c = soil cohesion were carried out by the Minsk-22 computer).

In most cases the initial ultimate stress state is characterized by the internal swelling for the medium with the vertical perviousness prevalence.

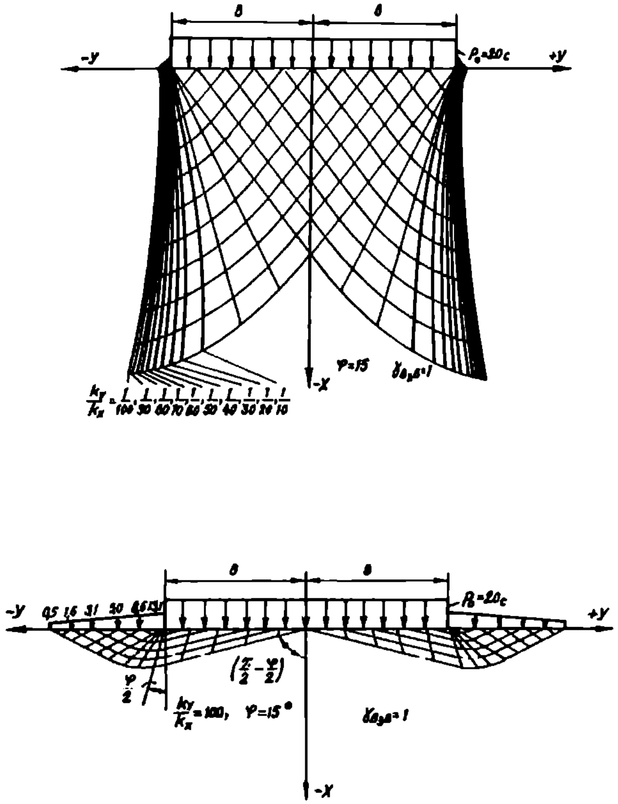


Fig.2. Slip nets for anisotropic pervious soils with transient loading which is $P_0=20c$

a) for soils with vertical permeability prevalence,
b) for soils with horizontal permeability prevalence.

To confirm a theoretical qualitative picture of deforming quickly loaded anisotropic saturated soils laboratory tests in a metal box were carried out. This box was filled with dry, fine sand in layers 3mm to 5mm and powder of Kaolinite clay in layers 10 to 12mm. By these tests the permeability coefficient ratio in mutually perpendicular directions was found to be 300 or $\frac{1}{300}$.

The soil was saturated from bottom and sides. Loading was carried out by means of a settlement plate to be 260 cm^2. 6 kg load was applied to the settlement plate with 1-2,5 m interval. The loading was made until a complete destruction occurred under the unit pressure of 0.1-0.25 kg/cm^2.

The deformation of the soil mass with the horizontal permeability prevalence is characterised by soil swelling in sides from the centre of the settlement plate. In this case no soil wedge was found.

The internal swelling deformation occurs in soils with the vertical permeability prevalence. The settlement of the plate was accompanied by some rise of the side surface without an obvious damage of the soil solidity. However, in the depth of the mass both sides of the wedge distinctly seen through the front glass wall had this damage.

The depth of the deformed zone was considerably greater than that of the medium with horizontal layers (fig.3).

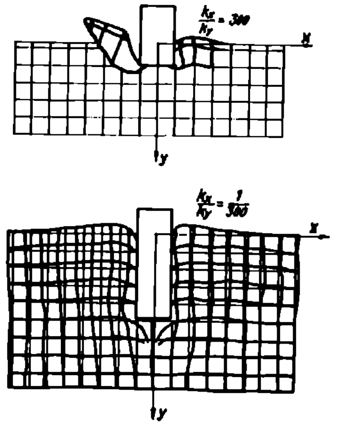


Fig.3. The nature of deforming models of anisotropic pervious soils.

a) for soils with vertical permeability prevalence

$$\frac{K_x}{K_y}=\frac{1}{300}\ ;$$

b) for soils with horisontal permeability prevalence

$$\frac{K_x}{K_y}=300$$

CONCLUSIONS

1. Filtration anisotropy influences, to a great extent, on the nature of deforming and stability of the quickly loaded saturated soils.

2. The stability loss is accompanied by the formation of the surface swelling for the saturated soils with the horizontal permeability prevalence.

3. A considerable settlement of the plate with the wedge and internal swelling is characteristic of to saturated soils with the vertical permeability prevalence.

Chairman Prof.Bengt B.Broms

Thank you Mr. Sobolevsky. Now I pass the work to Mr. Balissat from Switzerland

Mr. Marc Balissat (Switzerland)

<u>Synopsis</u>

A hydropower plant on the Reuss River (Switzerland) is located entirely on a silty subsoil. The design of the dam is completed by two cutoff walls in order to reduce the seepage flows and to assure the stability of the structure. For the analysis the apron was considered together with the cutoff walls as a rigid frame. Furthermore, the influence of the elasticity of the subsoil on the static forces at the tops of the walls was investigated.

1. INTRODUCTION

The hydropower plant of Bremgarten-Zufikon is located on the River Reuss, one of the most important rivers of the Swiss Plateau. At the site (elevation 375 m a.s.l.) the Reuss describes a series of meanders cut out of a pre-glacial moraine. The project envisages to rise the water level about 11 m by means of a 85 m long concrete dam. This dam will be composed of 5 independent spillway bays and one power house block with 2 bulb turbines. With a maximum discharge of 200 m3/s the installed capacity will be 18 MW and the energy production will reach about 100 million kWh per year. The plant should be ready for operation in 1975.

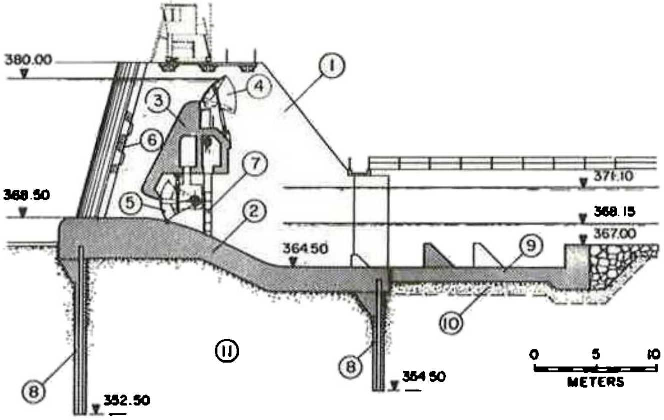


FIG. 1. LONGITUDINAL SECTION OF A SPILLWAY BAY

2. GEOTECHNICAL DATA

The subsoil is constituted by a thick moraine from pre-glacial times, the exact depth of which is unknown. None of the borings which were drilled to a maximum depth of 75 m reached the bedrock. The moraine contains dense to very dense materials without any orderly stratification. The most frequent soils are silts with low plasticity (ML) more or less sandy (SM - ML) with an important percentage of stones and boulders. Furthermore, there are thin intercalations of clay (CL) as well as sand and gravel (SM, GM). The average permeability k of the subsoil varies in the range from $2 \cdot 10^{-7}$ to $5 \cdot 10^{-5}$ cm/s. In certain fine sand deposits (SM - ML) k reaches $2 \cdot 10^{-4}$ cm/s.

Standard penetration tests as well as sheet piles penetration tests have shown that the density of the subsoil is quickly increasing with the depth. Interesting too is the low compressibility of the clayey and silty material (CL - ML) extracted at depths of 5 to 17 m. Their void ratio e varies from 0.425 to 0.700 and their compression index C_c from 0.042 to 0.082.

LEGEND

1	Pier
2	Apron
3	Intermediate beam
4	Flap gate
5	Radial gate
6	Upstream stop logs
7	Downstream stop logs
8	Cut-off walls
9	Stilling basin
10	Drainage blanket
11	Morainic deposits

3. FOUNDATION DESIGN

Each element of spillway is 11.80 m wide. It is composed of a thick apron, framed by two half-piers with an opening of 8.80 m (see Fig. 1.). A beam which supports a flap gate is located between the piers. Under this beam are openings closed by radial gates. In this way each element constitutes an independent stiff structure.

3.1 Cutoff Walls and Drainage Blanket

For the stability of the structure as well as in order to reduce the seepage flows under the apron, it was necessary to provide two cutoff walls upstream and downstream from the dam (A and B). The depth of the walls was determined by means of flow nets for equal and different permeabilities in horizontal and vertical directions, k_h and k_v. The stilling basin is independent from the rest of the structure in order to allow differential settlements. Under this basin a drainage blanket is provided to relieve water pressure and to prevent piping. Seepage water is estimated to be very low, i.e. 0.6 l/min. per bay assuming a permeability of $k = 2 \cdot 10^{-5}$ cm/s. Furthermore, the blanket will reduce the uplift pressure of 0.3 kg/cm2.

In view of the high density of the subsoil and the existence of large boulders, the following solution was adopted for the construction of the cutoff walls: joined bored piles with a diameter of 90 cm each in which sheet piles (New Larssen 22 section) are placed. In order to assure sufficient stiffness for the stability against sliding, the tops of the sheet piles will be fitted into the apron.

3.2 Method of Analysis for the Apron

For the calculation of the apron, the assumption was made that together with the cutoff walls it forms a rigid frame resting on elastic supports. The supports were materialized by values of compressibility modules $M_E = f \cdot \frac{\Delta\sigma}{\Delta h} \cdot D$ (where f is a form factor and D the smallest dimension of the foundation). The supports were introduced horizontally along the cutoff walls as well as vertically under the apron, taking into account different values of M_E in order to reproduce the variation of density with the depth. The calculation was made by means of a STRESS-computer program. The results are summarized in Fig. 2 for two cross-sections near the tops of the cutoff walls. The assumption of an even not very deformable elastic support, at the bottom of the walls entails an important reduction of the vertical forces (V'_A and V'_B).

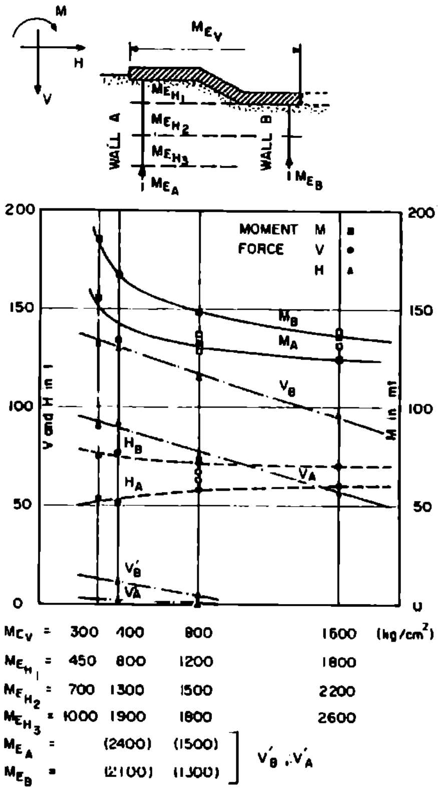


FIG. 2. INFLUENCE OF COMPRESSIBILITY MODULUS M_E ON THE FORCES V AND H AND THE MOMENT M AT THE TOPS OF CUTOFF WALLS A AND B

Chairman Prof. Bengt B.Broms

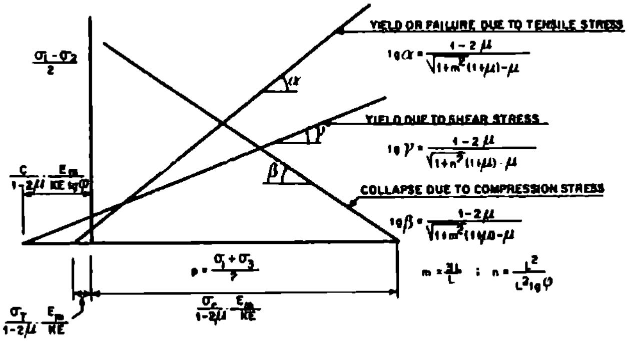
Thank you Mr. Balissat. Now we shall listen to Mr. Uriel from Spain. Mr. Uriel will you, please.

Mr. S.Uriel (Spain)

GEOTECHNICAL PROPERTIES OF TWO COLLAPSIBLE VOLCANIC SOILS OF LOW BULK DENSITY AT THE SITE OF TWO DAMS IN CANARY ISLAND. (SPAIN). S. Uriel and A. A. Serrano.

Among the several types of collapsible soils, this report deals with those which has a sudden change - stress of the apparent modulus of deformation when the effective stresses reach a certain level, due to - the failure of the bonds between the particles of the - soil. This is the case of cemented clays or silts, - rocks of a great porosity, volcanic ashes, etc, which collapses when summited to increasing stress field.

In the figure 1 are indicated in a theoretical p-q diagram the conditions which produces the collapse of - one soil of this type. The collapse can be originated by the failure of the bonds in three different ways: by tensile, shear and compression stress. Failure by - shearing stress can take place if the bonds are short and wide. On the contrary if the bonds are long and - slender only collapse by tension or compression is - possible. The formulas indicated in figure 1 refer to homogeneous and isotropic soil. If the cementation - presents preferential orientations, the relative position of the lines p-q would be different it being possible that one of the three types of failure can predominate.



l = MEAN WIDTH OF THE BONDS
L = MEAN LENGTH OF THE BONDS
E_m = APPARENT DEFORMATION MODULUS OF THE SOIL
μ = POISSON RATIO OF THE SOIL
E = DEFORMATION MODULUS OF THE BOND MATERIAL
K = SEE TEXT
σ_c = COMPRESSIVE STRENGTH OF THE BOND MATERIAL
σ_T = TENSILE STRENGTH OF THE BOND MATERIAL
c = COHESION BETWEEN SOIL PARTICLES AND BONDS
φ = FRICTION ANGLE BETWEEN SOIL PARTICLES AND BONDS

FIG. 1

In the figure 2 and 3 are presented the result obtained in triaxial tests for two volcanic cemented agglomerates of the Canary Island, which will be the foundation soil of two dams: Los Campitos and Ariñez dams. It can be easily seen the tensile and compresive type - of failure, although the tests are few for clearly ob - served, if it exists, the shear stress failure.

The volcanic soils of the Los Campitos dam, having a bulk density between 0.7 and 1.0, seem to follow a law similar to the one just indicated, although there are not enough data to establish correlation between the different types of failure.

For the volcanic conglomerate of the Ariñez dam, - which has a higher density (1.2 to 1.4 Tons/m³), - collapse takes place when the major principal stress reaches a value ranging from 27 to 35 kg/cm², due probably to an anisotropic distribution of the bonds.

For the Los Campitos dam the solution finally adopted was the removal of the material; For the Ariñez dam the major principal stress was required not to exceed 5 kg/cm².

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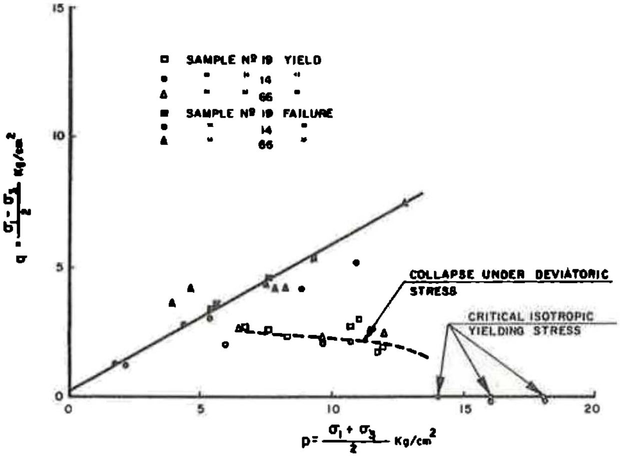

FIG.2.- p-q DIAGRAM FOR q_c AND q_f. LOS CAMPITOS DAM

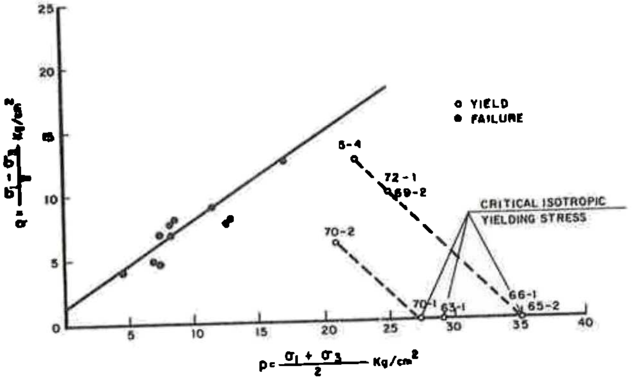

FIG.3 .- ARINEZ DAM p-q DIAGRAM FOR q_c AND q_f

Chairman Prof. Bengt B.Broms

Thank you Mr. Uriel for your interesting discussion. Now I pass the word to Mr. Herzog (Hungary)

Mr. Henrik Herzog (Hungary)

SETTLEMENT ANALYSIS OR PREDICTION OF MOVEMENTS OF THE GROUND. CONCLUSIONS TO BE DRAWN FROM A SERIES OF MEASUREMENTS

Over the area of the river barrage under construction at Kisköre, Hungary, movements of the ground have been observed ever since the construction work started.+

The river barrage comprises the power plant /4 units, 28 MW/, the weir /5 spans of 24 m each/ and the shiplock /12 by 85 m clearance/, one attached to the other. The area containing the structure is 230 by 150 m in size, with a maximum foundation depth of 22 m. On the upstream side a concrete diaphragm wall has been built, it reaches to a depth of about 20 m. The entire structure is constructed over the flood plain; after construction, water will be closed. The subgrade of the foundation consists of Pleistocene sediments: alternating layers of silty fine sand and clay, down to depths exceeding 200 m. The ground water was lowered in multiple stages.

Part of the bench marks involved in the observation of ground motions had been, placed in boreholes below the reference plans, before the excavation. Other bench marks were placed on independent foundation blocks on berms of the excavation pit, on the flood protection cofferdams, and also in areas remote from the construction site.

The results of measurements showed that

- during the excavation of the working pit the bench marks placed in depths went up to considerably higher elevations /they rose by 5 8 cm/;
- though motions occurring after the beginning of concrete pouring generally bear the character of settlement, after longer intervals /a few weeks/ in concreting, elevations from 0,5 to 1,5 cm may also occur;
- bench marks on the cofferdam placed at a depth of 20 m suffered settlement without exception /3 4 cm/;
- bench marks placed on a berm in adepth of 7 m below the original ground level showed motions of a few mm in alternate senses;
- vertical movements under the influence of construction activity /excavation of the pit, groundwater lowering, building of the structure, back filling, etc./ did not accur beyond 300 m from the structure;
- the structure showed, after groundwater lowering had been discontinued, an elevation from 1 to 2 cm, and about the same amount of subsidence after the river was led into the new channel.

Although the results of measurements permitted to give predictions on the basis of which the structure floors having an operational importance could be located - within permissible tolerances - on to the designed elevations, quantitative conclusions can not be transferred to other structure. However, some qualitative statements can be made, These are:

- vertical movements of structures depend on stress changes as related to the natural conditions;
- expansion accompanying a stress decrease is a time-dependent process.

+Investor: National Investment Agency for Agency for Hydraulic Projects, OVIBER, Budapest
Designer: Institute for Hydraulic Planning, VIZITERV, Budapest
Building contractor: Enterprise for Hydraulic Construction, VIZÉP, Budapest
The observations are being made by: Research Institute for Water Resources Development, VITUKI, Budapest

It can be concluded that the predictions of movements can be reliable only if the sequence and the time schedule of the activities construction, /excavation, dewatering concreting, etc./ the extension of layers affected by stress changes as well as their compression and expansion coefficients, are known.
Not all of these conditions can be fulfilled at present To fulfil them all, further investigations are required e.g. the physics of the expansion of soils. A more precise knowledge of stress distribution at depths would be also necessary, a special consideration being given to the interaction of factors increasing and diminishing stresses /e.g. the influence exerted by sheetings and diaphragm walls on the distribution of stresses, the effect of pressure level fluctuations of deeper lying artesian waters and of pit dewatering, etc./
Not before we are in possession of knowledge as indicated above shall we be in a position to draw the limits within which conventional methods of calculating settlements can meet the requirements of practice, that is beyond which an analysis of the entire process will be required.

Chairman Bengt B.Broms

Thank you Mr. Herzog for your contribution.

Now I should like to call on M.Viggiani from Italy. Mr. Viggiani will you please.

Mr. Viggiani (Italy)

SETTLEMENT OF A WEIR DAM DUE TO CONSOLIDATION INDUCED BY SEEPAGE FORCES.

In fig. 1 a schematic section of the Caprizi weir dam on the river Tagliamento, in Italy, is shown; the average subsoil constitution and some typical soils properties are also reported.

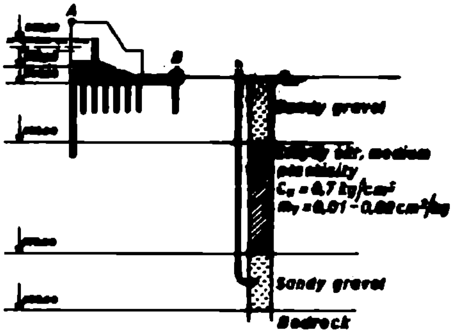

Fig. 1. Schematic section of the Caprizi weir dam and its subsoil. Figures shown are elevations in meters above sea level

The behavior of the dam during operation has been characterized by a slow settlement accompanied by a progressive tilt toward the reservoir. In fig. 2 the observed settlement of two points of the dam are reported; the measurements cover a period of 16 years after the end of construction. Between 1960 and 1965 the settlement rate is affected by grouting work carried out in the gravel underlying the dam in the period shown in fig. 2; anyway, after 1965, the effects of this perturbation may be seen to have practically vanished.

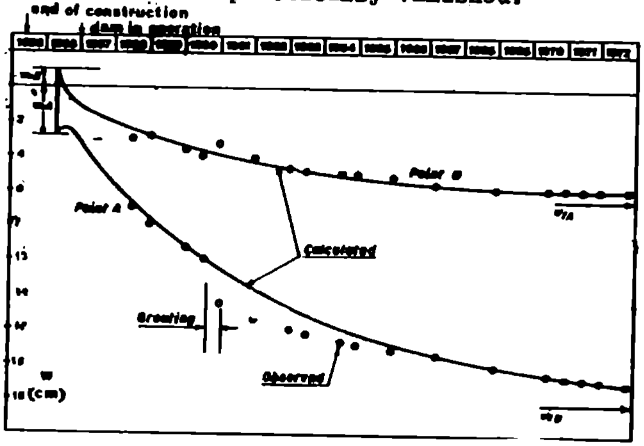

Fig. 2. Calculated and observed time-settlement behavior of two points of the dam. The location of points A and B is reported in fig. 1.

An examination of the problem revealed that the observed behavior is controlled by the consolidation of the clay layer under the action of seepage forces. With the occurring boundary conditions such forces produce a compression of the clay located upstream the cut-off and a swell downstream. As a consequence, the dam tilts toward the reservoir like a retaining wall founded on compressible soils tilts toward its fill.
The settlement-time behavior has been interpreted by means of some simple calculations. The total stress, the immediate settlement w_o and the undrained pore pressure have been calculated by means of the elastic theory, while the final steady state pore pressure distribution has been obtained by seepage theory; the clay layer was assumed homogeneous and isotropic.
The two-dimensional pore pressure dissipation has been calculated by numerical treatment of the simple Terzaghi-Rendulic diffusion type theory (Davis, Poulos, 1972) the final settlement w_f by means of the oedometer method.

Details of the calculations are reported elsewhere (Viggiani, 1973); in fig. 2 the calculated time-settlement curves are superimposed to the observed points. The best fit is obtained with $c_v=2.1\times10^{-3}$ cmq/sec; laboratory values of the consolidation coefficient range between 2.3 and 4.5×10^{-3} cmq/sec.

The surprisingly good agreement between such a simplified analysis and the observed behavior may be largely fortuitous; nevertheless it is believed that this case history demonstrates the importance of a proper consideration of seepage forces. In the present case these forces are responsible for the unexpected upward tilt of the dam under a water thrust tending to rotate it in the opposite sense.

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Chairman Prof. Bengt B.Broms.

Thank you Mr. Viggiani.

Now I want to pass the word to Mr. Kulikov from the USSR. Mr. Kulikov will you please.

Kulikov K.K. (USSR)

OPTIMIZATION OF BASEMENT WORK UNDER THE CONDITIONS OF PLAIN STRAIN

Development of the experimental technique and vast investigations of the strain distribution and the basement bearing ability carried out by Evdokimov P.D. Malyshev M.V. Lipovetskaja T.F. ,Krivorotov A.P., Vinokurov E.F., Murzenko U.N., Snirjaev R.A., Tarikuliev Z.J. and others created a modern experimental base and methodics for conducting complex investigations of basements and structures.

The aim of carrying out our researches was studying peculiarities of interaction mechanism of system "basement-structure" and revealing optimum conditions of basement work depending on the initial data of the experiment-flexibility,deepening.

Experimental investigation of work of the compact sand basement under the conditions of plain deformation were carried out in the laboratory "Basements and foundations" of the chair "Engineering structures" of the Novocherkassk Polytechnical Institute. These investigations were performed upon the universal testing machine MΦ-1 as well as in the field conditions using soil pressure metres and registering devices designed by Prof. Murzenko U.N.

50 experiments dealing with this theme upon the rigid and flexible structures (t_H=0 + 20 according to the Gorbunov-Posadov method) from 250 till 1200 mm in width with the relative deepening H: B=0.0+1.5 were carried out all in all.

Loading in the experiments was accomplished by means of the "VNIIG "a scheme mainly.

The results of the experimental researches indicated that in general case the theoretical conclusions concerning foundation work under conditions of plain deformation are not observed. Longitudinal epures of contact stresses (main criterion of the basements "plain strain") are not of a line character having vividly expressed wavy character, wave picks alternating in a multiple way to the structure width.

Load increasing, a successive transformation of alternating spatial seddle-shaped epures occured under the absolutely rigid structure. These epures transformed firstly into wavy ones, and then immediately under the critical load into completed pick-shaped attended by the edge ordinates' dropping (points 1, 1-a, 1-b Fig.1).

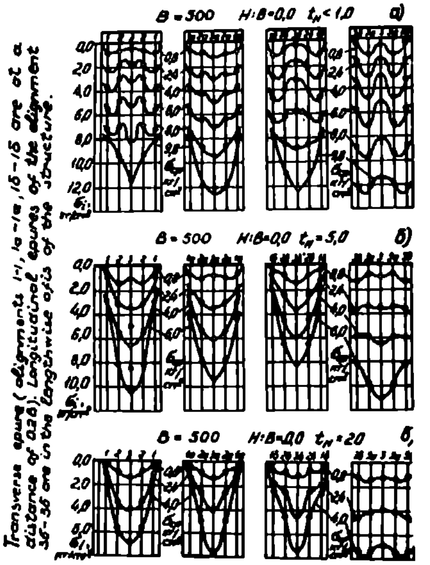


Fig.1

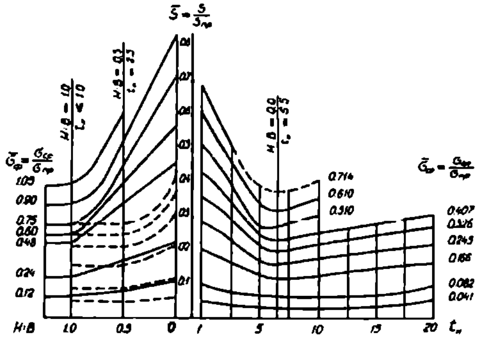
With the increasing of the structures' initial flexibility (t_H=2,0+5,0) transformation of epures from seddle-shaped into wavy and bell-shaped ones was observed. The character of epures was influenced by structures consoles deflection, which excluded initial seddle-shaped form of the epures and accelerated the general process of transformation.

Measurements of the basement contact layer field of density under the load conditions below limiting showed the identity of distribution of contact stress and changing density in the contact layer, but under the conditions of limiting load they showed the identity of the density and the contact stress epures' outline with the outline of the fixed at that very moment limit resilient pick-shaped core.

Nonlinear character of longitudinal epures of the contact stresses gives rise not only to the consoles bending, but bending with torsion, what considerably complicates the working conditions of the "basement-structure" system and provokes longitudinal stretching stresses in the upper zone of the structure not taken into account during the calculations according to the existing theories.

Difference in the values of bending moments of the adjacent transverse ranges being at the distance of 0.2+0.4 B equals to 40+50%.

Investigation of influence of the flexible structure cutting into blocks (t_H=5.0) with different ratio of structure width to that of the block (in the plan) showed that noncut monolithic structure is optimum for the conditions of maximum using of bearing ability of a basement and a structure. But as far as conditions of the crack-resistance of a structure is concerned the structure made of blocks with the ratio 0.3 (in the plan) is optimum. Fig.2 denotes a dimensionless diagram of changing of settlements of uniform sand basement for various medium pressures in shears of the critical loading upon the basement for a rigid non-buried structure depending on the initial flexibility of the structure and deepening.



$\bar{S}_{cp}$ and $\bar{G}_{np}$ are accordingly relative meanings in shears of the limit meanings of S_{np} and G_{np} for nonburied rigid structures.

Fig.2

In general case non-buried flexible structure (1.0 < t_H < 15.0) when loaded undergo distortion of basement less than a rigid structure of different dimensions.

Transition from an absolutely rigid structure to the flexible ones with the figures t_H= 2.5 and 5.0 are characterized by decreasing of settlements relativly for 20 and 40%.

In the constructions having initial flexibility 10 and 20 for the account of great deflection of consoles the cross-section acquires a form of a crimple, favouring lessening the resistance of ground and increasing of settlements.

The character of dimensionless settlements changing depending on the initial flexibility permits to determine the optimum initial flexibility (t_H=6.5) of the structure, which being loaded experiences the least settlements.

With the increasing of the rigid and flexible structures relative deepening (H:B=0+1.5) general regularity of settlements' decreasing is observed.

Settlements of rigid structure at the relative deepening 0.5 and 1.0 were 20 and 40% less than those nonburied structure (like in the case of increasing flexibility in the nonburied experiments).

With the increasing of relative deepening in the interval 1.0-1.5 further diminishing of settlements was not observed.

The settlements during the deepening equal to H:B=1.0; 1.25 and 1.5 are practically equal. The same regularity was observed in the case of changing critical loads upon the basement. For instance, during the relative deepening 0.5 the meanings of critical loads are equal to doubled, but during the relative deepening 1.0 to trebled meaning of critical loads for nonburied rigid structure. Further increasing of deepening (H:B > 1.0) was not accompanied by so intensive rise of critical load. The latter reveals qualitative changing of influence of lateral increasing of load upon the supporting power and deformation ability of the basement beginning with deepening H:B=1.0.

Deepening H:B=1,0 is optimum (t_H < 1.0) from the conditions of minimum deformation-ability and maximum using of supporting power of the basement. For the structure flexibility 2.5 relative deepening 0.5 is optimum.

General analysis of the results of the investigations carried out by us showed that there are existing: optimum flexibility, deepening, cutting structures into blocks, which permits to apply supporting power of the basement in the best way for the account of the proper determination of the initial conditions of the basement work.

Chairman Bengt B. Broms
Thank you very much Mr. Kulikov.
The next will be Mr. Jakovlev (USSR)

P.I. Jakovlev (USSR)

ON PRACTICAL METHODS FOR CALCULATION OF BEARING CAPACITY OF FOUNDATIONS WITH COMPLICATED BOUNDARY CONDITIONS

The calculations made for designing of real foundations are always practically connected to the necessity to consider complicated boundary conditions. For example, while calculating marine hydrotechnical structures one should in all cases consider complicated tensile loads, multilayer foundations and stability of slopes under inclined uneven stretched load applied at a certain distance from the edge, etc.

Currently, there are suggested many empiric methods of foundation strength calculation based on different subjective assumptions and some experimental data. Naturally, these methods cannot evaluate the influence of applied assumptions on the final results. On the other hand the methods based on a series of experiments cannot be successfully applied to the calculation of constructions with different boundary conditions.

The most promising way for solution of various practical problems is to develop approximate engineering methods of calculation based on a general theory. Coulomb's theory is an example of the viability of such approach. Hundreds of scientists and engineers using extreme principles of this theory and making some additional assumptions got engineering solutions practically for every problem encountered. These methods were successfully applied in the course of many decades and are still used in engineering practice.

The problems relating to bearing capacity of foundation are presently solved using the general theory of limiting balance elaborated by soviet scientists V.V.Sokolovsky, S.S.Golushkevich, P.D.Evdokimov (1956), F.M.Shikhiev and others. The engineering methods of calculation making account of the most complicated boundary conditions can be developed on a basis of this theory. In the USSR such methods had been developed for the cases of multilayer foundations, seismic activity, deeply laid foundations, etc (Jakovlev, 1965, 1972). The application of foundamental principles of safe stress state theory enables to achieve satisfactory results when solving the most complicated practical problems.

It should be noted that the theory of limiting balance treats equally all problems arising in the process of interaction between structure and soil, which is its great advantage.

In the USSR on a basis of this theory there have been developed engineering methods for calculation of back-fill pressure on retaining walls at different boundary conditions which are applied for solving various problems relating to stability of slopes and other fields (Shikhiev, Jakovlev, 1972).

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Chairman Prof. Bengt B.Broms

CONCLUDING REMARKS

The specialty session on "Soft Soil Bases of Concrete Hydrotechnical structures" focused the attention on the effects of progressive failure and liquefaction on the foundation strength and the stability of gravity and buttress dams, offshore structures, locks, drydocks and ligthouses. It was pointed out that the risk of progressive failure must be considered in the design of such structures particularly when they are founded on overconsolidated plastic clays. A reduced shear strength between the peak and the residual strength should be used.

It was also pointed out that earth quakes, blasting and wave forces can cause liquefaction in fine uniform sand. Liquefaction occurs when the pore pressure increase caused by cyclic loading approach the initial vertical effective stress in the soil. The development of liquefaction is dependent on such factors as the shear stress ratio in the soil, the initial relative density and on the drainage conditions.

Attention was also focused at this specialty session on the use of the finite element method for the prediction of settlements and lateral displacements. With the finite element method it is possible to study the effects of such factors as creep and consolidation. The success of the finite element method is, however, dependent to a large extent on how accurately the soil parameters used in the analysis can be determined. It is necessary to pay more attention to the development of in situ testing methods for an improved prediction of these parameters.

Hydrotechnical structures are frequently large. Size effects becomes important when the results from small scale test are analyzed and applied as pointed out by Mazurkiewicz. He found that the measured settlements of a dry dock were considerably smaller than those predicted from plate load tests or from oedometer tests.

Giroud discussed the rotation of rigid slabs founded on a compressible elastic foundation. The results were presented in the form of nomographs. Giroud has investigated also the area (core) within which a vertical load will only cause compressive stresses below flexible or rigid slabs.

Samarin presented a method to calculate the lateral displacement of rigid structures by considering first the deformations at the time of loading and then the time dependent deformations.

Fukuoka described the design of the Sabaishigawa dam in Japan which has been constructed on soft sandstore and mudstone.The strength and deformation properties of the foundation materials were evaluated by insitu direct shear tests.The dam and the foundation was analyzed by the finite element method.

The design of a concrete dam founded on silt was discussed by Balissant. The dam was provided with two cut off walls and an apron to reduce seepage below the structure. The cut off and the apron were analyzed as a rigid frame supported on elastic springs.

Savey described the design of a 265 m long lock which has been constructed on clay,clayey silt and peat. The shear strength of the soil was increased by preloading. Sand drains, reinforced with fibre glass wicks, were used to increase the consolidation rate. The observed settlements were larger than the calculated settlements.

WRITTEN CONTRIBUTIONS

FOUNDATION OF A RIVER DAM ON SILTY SUBSOIL.

Marc Balissat /Switzerland/

SYNOPSIS

A hydropower plant on the Reuss River (Switzerland) is located entirely on a silty subsoil. The design of the dam is completed by two cutoff walls in order to reduce the seepage flows and to assure the stability of the structure. For the analysis the apron was considered together with the cutoff walls as a rigid frame. Furthermore, the influence of the elasticity of the subsoil on the static forces at the tops of the walls was investigated.

1. INTRODUCTION

The hydropower plant of Bremgarten-Zufikon is located on the River Reuss, one of the most important rivers of the Swiss Plateau. At the site (elevation 375m a.s.l.) the Reuss describes a series of meanders cut out of a preglacial moraine. The project envisages to rise the water level about 11 m by means of a 85 m long concrete dam. This dam will be composed of 5 independent spillway bays and one power house block with 2 bulb turbines. With a maximum discharge of $200 m^3/s$ the installed capacity will be 18 MW and the energy production will reach about 100 million kWh per year. The plant should be ready for operation in 1975.

2. GEOTECHNICAL DATA

The subsoil is constituted by a thick moraine from preglacial times, the exact depth of which is unknown. None of the borings which were drilled to a maximum depth of 75m reached the bedrock. The moraine contains dense to very dense materials without any orderly stratification. The most frequent soils are silts with low plasticity (ML) more or less sandy (SM-ML) with an important percentage of stones and boulders. Furthermore, there are thin intercalations of clay (CL) as well as sand and gravel (SM, GM). The average permeability k of the subsoil varies in the range from 2.10^{-7} to 5.10^{-5} cm/s. In certain fine sand deposits (SM-ML) k reaches 2.10^{-4} cm/s. Standard penetration tests as well as sheet piles penetration tests have shown that the density of the subsoil is quickly increasing with the depth. Interesting too is the low compressibility of the clayey and silty material (CL-ML) extracted at depth of 5 to 17m. Their void ratio e varies from 0.425 to 0.700 and their compression index C_c from 0.042 to 0.082.

LEGEND

1	Pier
2	Apron
3	Intermediate beam
4	Flap gate
5	Radial gate
6	Upstream stop logs
7	Downstream stop logs
8	Cut-off walls
9	Stilling basin
10	Drainage blanket
11	Morainic deposits

3. FOUNDATION DESIGN

Each element of spillway is 11.80m wide. It i is composed of a thick apron, framed by two half-piers with an opening of 8.80m (see Fig.1). A beam which supports a flap gate is located between the piers. Under this beam are openings closed by radial gates. In this way each element constitutes an independent stiff structure.

3.1. Cutoff Walls and Drainage Blanket

For the stability of the structure as well as in order to reduce the seepage flows under the apron, it was necessary to provide

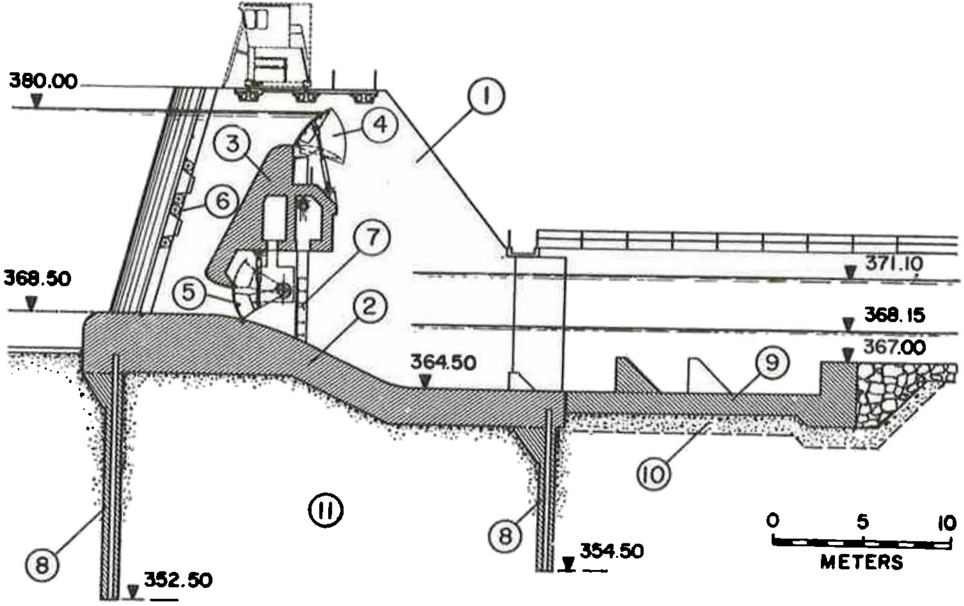


FIG. 1. LONGITUDINAL SECTION OF A SPILLWAY BAY

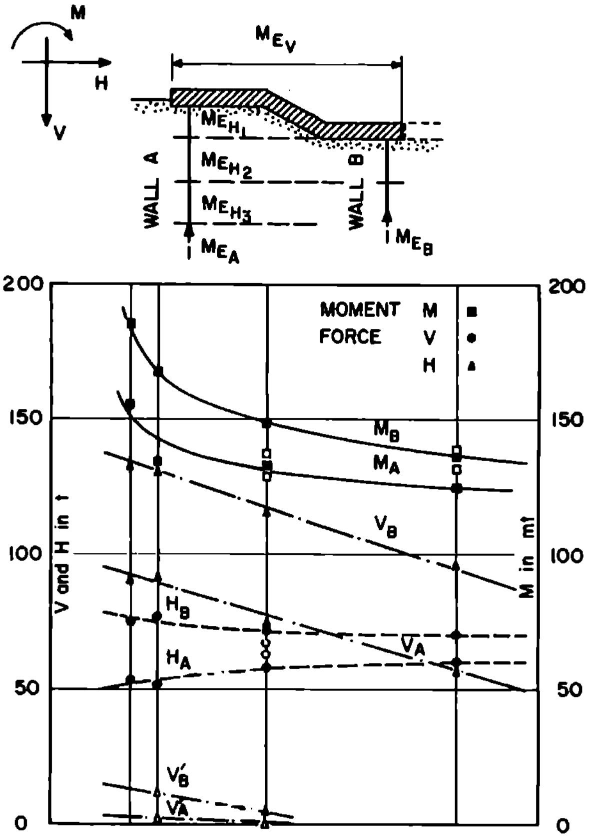
two cutoff walls upstream and downstream from the dam (A and B). The depth of the walls was determined by means of flow nets for equal and different permeabilities in horizontal and vertical directions, k_h and k_v. The stilling basin is independent from the rest of the structure in order to allow differential settlements. Under this basin a drainage blanket is provided to relieve water pressure and to prevent piping. Seepage water is estimated to be very low, i.e. 0.61/min. per bay assuming a permeability of $k=2.10^{-5}$cm/s. Furthermore, the blanket will reduce the uplift pressure of 0.3 kg/cm^2.

In view of the high density of the subsoil and the existence of large boulders, the following solution was adopted for the construction of the cutoff walls: joined bored piles with a diameter of 90cm each in which sheet piles (New Larssen 22 section) are placed. In order to assure sufficient stiffness for the stability against sliding, the tops of the sheet piles will be fitted into the apron.

3.2. Method of Analysis for the Apron

For the calculation of the apron, the assumption was made that together with the cutoff walls it forms a rigid frame resting on elastic supports. The supports were materialized by values of compressibility modulus

$M_E = f.\frac{\Delta \sigma}{\Delta h}.D$ (where f is a form factor and D the smallest dimension of the foundation). The supports were introduced horizontally along the cutoff walls as well as vertically under the apron, taking into account different values of M_E in order to reproduce the variation of density with the depth. The calculation was made by means of a STRESS-computer program. The results are summarized in Fig. 2 for two cross-sections near the tops of the cutoff walls. The assumption of an even not very deformable elastic support, at the bottom of the walls entails an important reduction of the vertical forces (V'_A and V'_B).



M_{E_V} =	300	400	800	1500	(kg/cm^2)
$M_{E_{H_1}}$ =	450	800	1200	1800	
$M_{E_{H_2}}$ =	700	1300	1500	2200	
$M_{E_{H_3}}$ =	1000	1900	1800	2600	
M_{E_A} =		(2400)	(1500)	V'_B, V'_A	
M_{E_B} =		(2100)	(1300)		

FIG. 2. INFLUENCE OF COMPRESSIBILITY MODULUS M_E ON THE FORCES V AND H AND THE MOMENT M AT THE TOPS OF CUTOFF WALLS A AND B

USE OF COMPLEXES OF FIELD METHODS IN STUDIES OF EARTH FOUNDATIONS OF HYDRAULIC STRUCTURES

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Studies of soft soil foundations by a complex of field methods on a basis of the results of large-scale studies of soils by express methods and those of a limited member of detailed studies of soils in typical ("key") places of their occurrence allow for improving the reliability and thoroughness of studies.
for curring down the time of carrying out: the studies and reducing the cost of designing, research and construction work. The filed methods can be divided into three main groups:

Group I - convetional methods of direct determining of the design characteristics of the mechanical properties of soils and the pile capacity (E, φ, C, P pile);

Group II - methods which require preliminary calibration to define the physical state and design characteristics of soils;

Group III - methods of indirect determining of the soil state and properties from the correlation relations. Most of the methods placed into Groups I and II, require the preliminary driving of underground workings;

Groups II and III cover mainly the methods of accelerated tests of soils (express-methods) which are standardized and there is special standard equipment produced in the USSR to carry out these tests.

When working out the methods of field studies the authors proceeded from the to day knowledge of the nature of the physical and mechanical properties of soils, their facies variation and from the results of the theoretical and experimental studies of the physics of phenomena observed in the soils during the tests when different methods are applied.

Based on the acquired experience the following procedure of studies of soft soil foundations of hydraulic structures using a complex of field studies methods is recommended below:

1. Analysis of the data available of the construction area to effect its preliminary engineering and geological zonation.
2. Choice of a complex of field methods depending on the soil type and design particulars of the proposed structures.
3. Large-scale soil studies by express-methods parallel with a limited number of test boreholes drilled.
4. Choice of a limited amount of "key" places typical for the area considered based on the above data and detailed studies of the soils using a complex of selected field methods.
5. Intercorrection of the data obtained in "key" places by different methods and defining the calibration relations for methods entering Groups II and III.
6. Application of the results of the detailed studies of soils in "key" places over the whole area under study by the express-methods.

The optimum combination of the amount of "key" places and places of soil studies and testing by express-methods is dictated by designing stage, complexity of engineering and geological conditions, the ares of the territory under study and by the design features of the proposed structures.

The schematic classification of the field methods and complexes of methods see in the Table given below.

Soil properties to be defined	Field Methods Groups			Complexes of field methods			
	I	II	III	Sand soil		Clay soil	
				A	B	A	B
Physical		PK^{x}ПКК	D^{3}C3	D3	PK(ПКК)	C3 orПКК	PK
Mechanical compressibility	Шт^{x}	ПМ^{x}	D3C3	D3(C3)		C3 of ПКК(D3)	Шт,ПМ
-shear strength	CЦ^{x}	BC^{x}	D3C3	D3(C3)	BC-in loose soils CЦ-in stabilized soils	C3 or ПКК	C,BC
-dynamical stability	KB^{x}	BШт	D3	D3	KB,BШт	-	-
Pipe capacity	CиC DиC	MC^{x}	C3(D3)	C3(D3)	MC DиC(CиC)	C3 or ПКК (D3)	Mo CиC (DиC)

Notes: 1) A - express-methods of large-scale testing of soils; B - a comples of methods for "key" places;
2) A possible alternative methods is indicated in brackets;
3) Physical properties of soils by direct methods are defined in laboratories.

NOTATION: Шт-stamp; CЦ - pillar shearing test; caving and building of prisms; KB - "camouflet" blast; CиC and DиC - static and dynamic pile tests; PK - radiation logging; ПКК - penetration and logging complex tests; (C3+PK); ПМ - tests by pressure meter; BC - vane tests; BШт - vibrating stamp; D3 and C3 - dynamic and static sounding; MC - pile model; x - methods which require preliminary drivinf of underground workings.

CALCUL DE LA ROTATION DES CONTREFORTS DE BARRAGES EN BETON SUR SOL NON ROCHEUX.

J.P. Giroud et J. Garnier (France)

Parmi les ouvrages hydrotechniques en béton sur fondations non rocheuses, on rencontre de nombreux barrages à contreforts. Ceux-ci subissent une rotation du fait de la charge inclinée et excentrée qu'ils exercent sur le sol. Dans le domaine des petites déformations, la théorie de l'élasticité permet, en supposant une distribution linéaire des réactions sous la fondation, de calculer la rotation moyenne. Nous montrons alors que, si l'excentricité et l'inclinaison de la charge respectent une certaine relation, la base ne subit aucune rotation. Cette relation fait intervenir le rapport des côtés du rectangle et le coefficient de Poisson du sol. Un abaque (Fig.1) permet de l'obtenir immédiatement. Ainsi, connaissant deux des trois paramètres (excentricité, inclinaison et rapport des côtés), on en déduit le troisième pour qu'il n'y ait pas de rotation de la base. Ce résultat a un intérêt pratique, car de petites rotations du contrefort entraînent de grands déplacements en crête du barrage.

Mais, du fait de la rigidité du contrefort, la distribution linéaire des réactions n'est qu'une hypothèse approchée. Dans le cas d'une charge excentrée normale, la rotation calculée en tenant compte de la grande rigidité du contrefort est plus faible de 15 % que dans le cas d'une répartition linéaire des réactions.

L'excentricité de la charge a une limite, le noyau central : le point d'application de la charge doit s'y trouver pour éviter l'apparition de contraintes de contact négatives (tractions). Avec l'hypothèse de la distribution linéaire des contraintes de contact, le noyau central a la forme d'un losange occupant le tiers central de la base. Si l'on veut tenir compte de la rigidité de l'ouvrage, la détermination du noyau central ne peut se faire que par une méthode numérique. Le calcul que nous avons fait, pour plusieurs valeurs du rapport des côtés de la base rectangulaire, montre que le noyau central d'une fondation rigide a une forme curviligne et qu'il est plus grand que le tiers central. L'exemple de la fondation carrée est donné sur la figure 2.

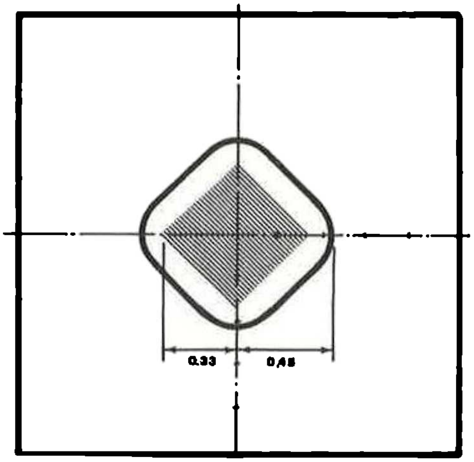

Fig. 2 Noyau central d'une fondation carrée rigide.

En conclusion, on retiendra deux résultats concernant la rotation des contreforts :

- le respect d'une certaine relation entre l'inclinaison et l'excentricité de la charge (compte tenu du rapport des côtés de la base du contrefort et des propriétés du sol) permet d'annuler la rotation du contrefort.
- le noyau central tel qu'il est habituellement déterminé (tiers central) est nettement plus petit que celui que l'on obtient en tenant compte de la rigidité du contrefort, ce qui est dans le sens de la sécurité.

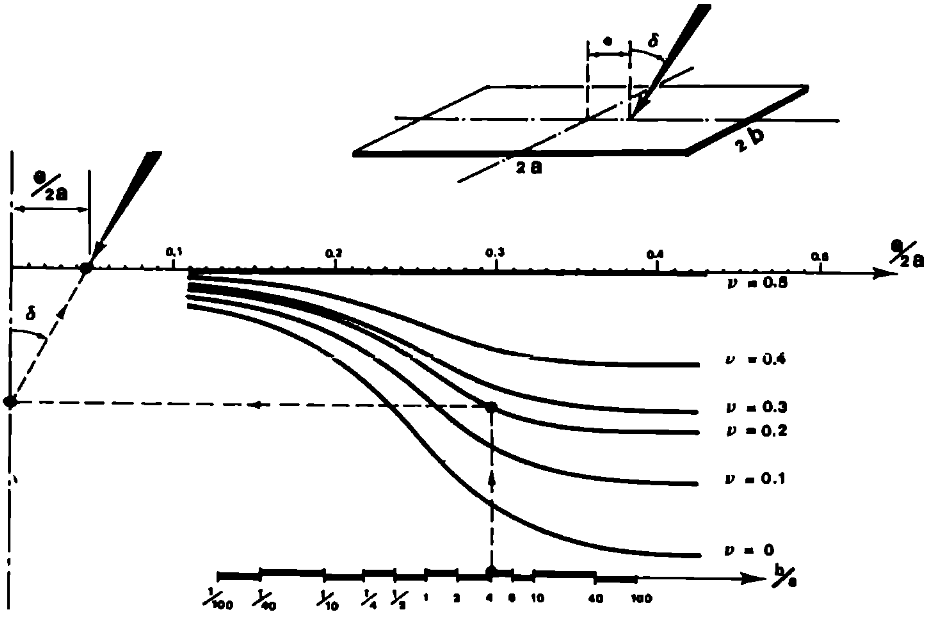

Fig. 1 Relation entre inclinaison et excentricité de la charge pour avoir une rotation nulle.

NATURE OF SAND STRENGTH AND SOME PROBLEMS OF EVALUATION OF SAND FOUNDATIONS OF HYDRAULIC STRUCTURES.

I.V.Doudler, E.F. Mosyakov, A.D.Potapov, V.A.Dourante (USSR)

It is customary to assume that, sand strength is mainly determined by its coarseness and porosity. Therefore, cohesion of sand when it gains in strength, its mineralogical composition, grain morphology and texture were not usually studied and evidently underestimated in civil engineering practice. These factors take account of a complex of conditions of sand deposits formation and that is whey sand of one and the same coarseness and porosity but different in genesis or occurring in different climatic zones are characterized by different strength and deformability.

This is approved by numberous studies carried out by soviet specialists during the last years in a number of regions of the Soviet Union.

In 1955 in the Soviet Union when taking dynamic sounding of alluvial quartz sand before and after disturbance of its structure it was first found that sand possess cohesion when gaining in strength.

During the ensuring years the soviet engineers studied the process of formation of water-resistant cohesion in artificially aggradated sand gaining in strength and estimated its positive impact on increase of dynamic stability, modulus of deformation and shearing strength   The results of these investigations were published in Transactions of Power Conference on soil Mechanics and Foundation Engineerings (V.A.Dourante & others, London, 1957; N.Y.Denissov, B.F.Peltov, Paris, 1962; N.Y.Denissov, I.V.Doudler, V.A.Dourante, M.I.Khazanov - Wiesbaden, 1963; N.Y.Denissov, I.V.Doudler, Oslo, 1967).

These and more recent investigations showed that along with the possibility of mobilization of sand cohesion in the foundation and body of earth filled structures under static loads, it should be borue in mind that dynamic effects lead to decrease of strength in sands with developped cohesion even in those cases when there is no sand liquescency and compression occurs to an extent.

Publication   gives appropriate recommendations on study and prediction of dynamic stability of sand and establishing increment in magnitude of possible earthquake intensity in seismic yoning.

Effect of grain morphology on the sand strength was noted in publications more than once. However, up to now attention was paid only to the grain shape. With the advent of electronic scanning microscope, it became evident that study of nature of grain surface holds a great promise   The investigations carried out showed, that 0.25 mm is a determining size in morphological studies carried out for civil engineering purposes.

Morphology of grains considerably effects on the texture peculiarities of sands which are also the functions of their strength and deformability   It is just differance in microstratification, orientation and degree of compactness of grains that is responsible for a pronounced difference in compressibility of equally-strong sand foundations made using different methods of sand filling. This fact was stated by Research Institute of Foundation

One important trend of studying sand strength nature is the study of chemical-colloidal processes on the surface of sand particles. Well known publications of the soviet (I.V.Grebenshikov, N.Y.Denissov, B.F.Reltov) and foreign scientists   prove promising value of these investigations. At present it can be said with confidence that the value of sand cohesion when it gains in strength due to colloid of silica can make up 0.2 - 0.3 kg per sq.cm. and even more.

Thus, when studying the foundation sands of hydraulic structures and earth-filled dams it is necessary to assess their genetic peculiarities, to take into account the possibility of mobiliyation of structural bonds under static loads and to foresee changes in sand strength after dynamic actions.

LOIS DE L'INTERACTION DE FORCE ET DE DEFORMATION ENTRE LES PLAQUES RIGIDES ET DE SOL DE FONDATION.

P.D. Evdokimov, T.F. Lipovetskaya, P.N. Kachkarov
(U.R.S.S.)

La similitude mécanique de l'état de contraintes dans les fondations sablonneuses ($C = 0$) des plaques rigides est assurée sous les conditions suivantes:

$$\frac{\alpha_\sigma}{\alpha_\ell \cdot \alpha_\gamma} = 1 ; \qquad \alpha_\varphi = 1 \qquad (1)$$

α_σ, α_ℓ, α_γ et α_φ étant respectivement les coefficients d'échelle pour les contraintes, les dimensions linéaires, le poids volumique et l'angle du frottement interne du sol de fondation [1]. Pour les conditions de la déformation bidimensionnelle les critères de similitude de l'état de contraintes d'une fondation sablonneuse des plaques rigides sont exprimés par les nombres de similitude N_σ et N_τ [2] :

$$\left.\begin{aligned} N_{\sigma_m} &= \frac{\sigma_{mn.\,m.}}{b_m \cdot \gamma_m} = N_{\sigma_n} = \frac{\sigma_{mn.\,n.}}{b_n \cdot \gamma_n} ; \\ N_{\tau_m} &= \frac{\tau_{mn.\,m.}}{b_m \cdot \gamma_m} = N_{\tau_n} = \frac{\tau_{mn.\,n.}}{b_n \cdot \gamma_n} ; \\ \varphi'_m &= \varphi'_n \end{aligned}\right\} \qquad (2)$$

Les résultats des recherches expérimentales sur l'interaction de force entre les plaques rigides de deux dimensions et la fondation en sable à grains moyens dans les conditions de la déformation bidimensionnelle sont donnés plus loin. Les méthodes expérimentales sont décrites dans les ouvrages [2,3]. La fig. 1 donne la courbe exprimant la relation entre les déplacements relatifs horizontaux des plaques $\frac{u}{b}$ (u - déplacement horizontal de la plaque, b - sa largeur) et le nombre de similitude N_τ pour les différentes valeurs de N_σ.

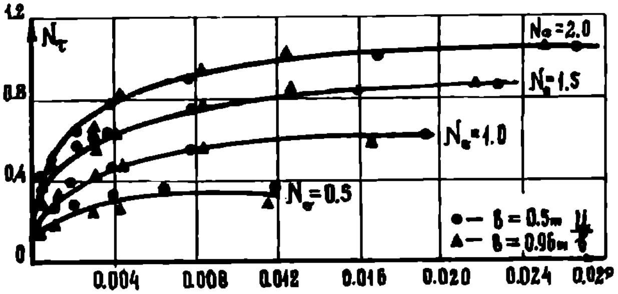

Fig. 1

La fig. 2 représente la courbe des relations entre $N_{\tau_{lim}}$ et N_σ construite selon les résultats expérimentaux en pleine conformité avec les critères de similitude [2]

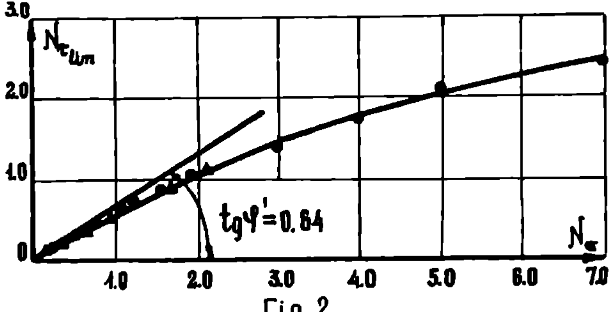

Fig. 2

La courbe exprimant les relations entre les déplacements relatifs horizontaux limites des plaques $\frac{u_{lim}}{b}$ (u_{lim} étant le déplacement horizontal de la plaque en état d'équilibre limite) et N_σ est donnée sur la fig. 3.

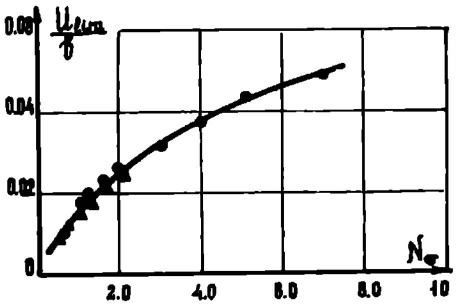

Fig. 3

Les résultats des essais relatifs à la détermination des contraintes normales de contact effectués sur les plaques de quatre dimensions ($b = 0.70$ m; 1.42 m; 1.75 m; 3.5 m) démontrent que les valeurs relatives des ordonnées correspondantes des diagrammes σ_0 des contraintes normales de contact $\frac{\sigma_c}{\sigma_{mn}}$ (σ_{mn} étant la valeur moyenne des contraintes dans la semelle d'une plaque) ne dépendent que de la valeur N_σ [3]. Les résultats expérimentaux justifient les critères de similitude (1) et (2). On les employait avec succès dans les recherches sur la capacité portante, les déplacements et l'état de contraintes des fondations sablonneuses des ouvrages, et, en particulier, pour la résolution du problème de contact. Comme l'on voit de la fig. 2, la courbe de la capacité portante des fondations sablonneuses a un tronçon linéaire qui caractérise la résistance au cisaillement spécifique (la capacité portante) τ_{lim} identique pour les différentes largeurs de la semelle d'une plaque et les mêmes valeurs de σ_{mn} se trouvant danc les limites des nombres critiques de similitude $N_{\sigma_{cr}}$ [2]

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CONCRETE GRAVITY DAM FOR FLOOD CONTROL CONSTRUCTED ON SOFT SOIL BASE. M.FUKUOKA (JAPAN)

What we call Sabo dams are constructed to prevent outflow of sand and gravel from the mountains during flood time. They are often constructed on soft bases composed of soft rocks, sand and gravel. There are many examples of damage to the dams caused by piping. Large cavities developed and cracks were observed in the dam. The design criteria based on the research and experience have been established as shown in Fig.1. The point of application of the resultant force is kept in the middle third of the base, when the design is made. Earthquake forces are neglected, but no damage has been reported yet.

Dams constructed for storing water have been designed much carefully. Dam sites having very good foundation for the construction, become comparatively few nowadays. Therefore, it is necessary to build a dam even on a soft soil ground, where no dam was contemplated before. It is quite natural to build a concrete dam instead of a fill dam, as latter is very weak against overtopping. Sabaishi Dam, now under construction, is one of the dams which is designed reflecting the above requirement. Fig.2 shows the plan of the dam. The soft soil base is composed of tertiary sandstone and mudstone. The stratum is almost level, but inclined slightly towards the downstream direction. Height of the main dam is about 37 m, and span is about 200 m. The blockshear tests, permeability tests, and grouting tests were performed. The mechanical tests were made on the samples taken from the dam site by drilling. The average values of the test results showed that $c=60\ t/m^2$ and $\phi = 40^\circ$ for mudstone and that $c=40\ t/m^2$, and $\phi = 30^\circ$ for sandstone. Coefficient of permeability of sand was $k=10$ cm/sec. Fig.3 shows the cross section of the main dam, the apron and the end sill. The thick concrete slab is constructed as the apron, and the end sill is placed at the end of the apron, in order to prevent scoring the overtopping water. Stresses in the dam and the base were analysed by the finite element method. Fig.4 shows the arrangement of the curtain grout to prevent the seepage water through the subsoil. The finite element method is applied to analyse the seepage flow. There are no reliable design criteria about the seepage flow, but perhaps the velocity of seeping water should be kept below 10^{-4} cm/sec. Consolidation grout under the dam is made in two series, namely primary and secondary. This dam is under construction as stated above, but the plan of grouting will be changed during the construction according to the result of observation.

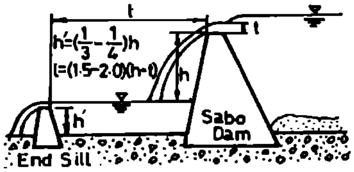

Fig.1 SABO DAM

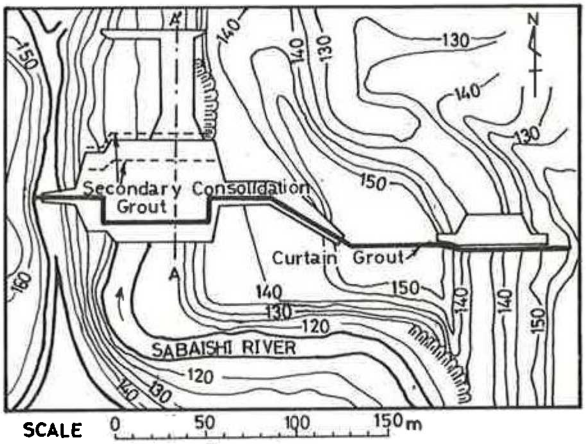

Fig.2 SABAISHI DAM

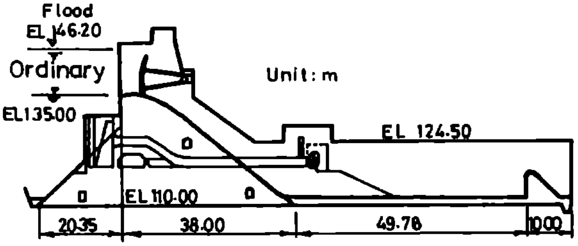

Fig.3 CROSS SECTION A-A'

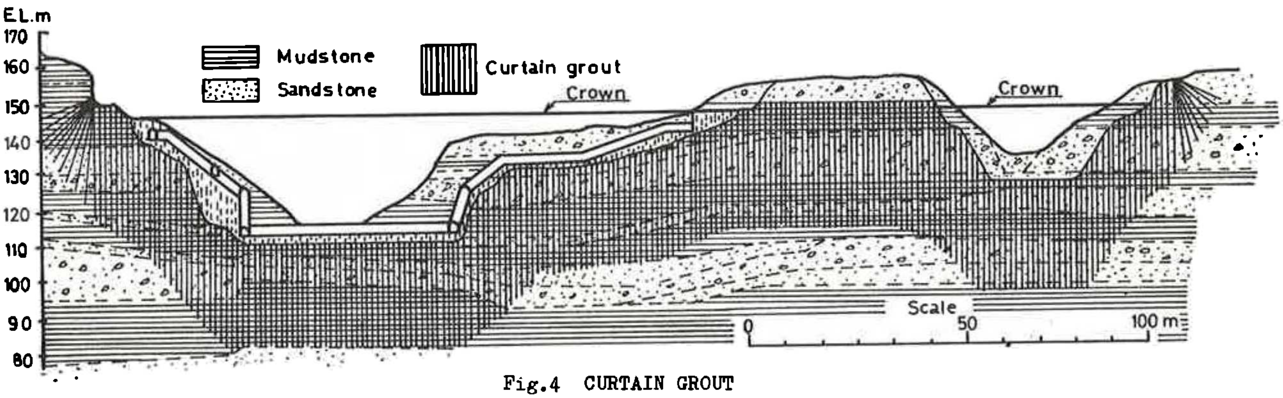

Fig.4 CURTAIN GROUT

SETTLEMENT ANALYSIS OR PREDICTION OF MOVEMENTS OF THE GROUND. CONCLUSIONS TO BE DRAWN FROM A SERIES OF MEASUREMENTS.

Henrik Herzog (Hungary)

Over the area of the river barrage under construction at Kisköre, Hungary, movements of the ground have been observed ever since the construction work started.+

The river barrage comprises the power plant /4 units, 28 MW/, the weir /5 spans of 24 m each/ and the shiplock /12 by 85m clearance/, one attached to the other. The area containing the structure is 230 by 150m in size, with a maximum foundation depth of 2 2m. On the upstream side a concrete diaphragm wall has been built, it reaches to a depth of about 20m. The entire structure is constructed over the flood plain: after construction water will be closed. The subgrade of the foundation consists of Pleistocene sediments: alternating layers of silty fine sand and clay, down to depths exceeding 200m The ground water was lowered in multiple stages.

Part of the bench marks involved in the observation of ground motions had been placed in boreholes below the reference plans, before the excavation. Other bench marks were placed on independent foundation blocks on berms of the excavation pit, on the flood protection cofferdams, and also in areas remote from the construction site.

The results of measurements showed that

- during the excavation of the working pit the bench marks placed in depths went up to considerably higher elevations /they rose by 5 8 cm/;
- though motions occurring after the beginning of concrete pouring generally bear the character of settlement, after longer intervals /a few weeks/ in concreting, elevations from 0,5 to 1,5 cm may also occur;
- bench marks on the cofferdam placed at a depth of 20m suffered settlement without exception /3 4cm/;
- bench marks placed on a berm in depth of 7 m below the original ground level showed motions of a few mm in alternate senses;
- vertical movements under the influence of construction activity /excavation of the pit, groundwater lowering, building of the structure, back filling, etc./ did not occur beyond 300 m from the structure;
- the structure showed, after groundwater lowering had been discontinued, an elevation from 1 to 2 cm, and about the same amount of subsidence after the river was led into the new channel.

Although the results of measurement permitted to give predictions on the basis of which the structure floors having an operational importance could be located-within permissible tolerances- on to the designed elevations, quantitative conclusions can not be transferred to other structure. However, some qualitative statements can be made, These are:

- vertical movements of structures depend on stress changes as related to the natural conditions;
- expansion accompanying a stress decrease is a time-dependent process.

It can be concluded that the predictions of movements can be reliable only if the sequence and the time schedule of the activities construction, /excavation, dewatering concreting, etc/ the extension of layer effected by stress changes as well as their compression and expansion coefficients, are known.

Not all of these conditions can be fulfilled at present. To fulfil them all, further investigations are required e.g. the physics of the expansion of soils. A more precise knowledge of stress distribution at depths would be also necessary, a special consideration being given to the interaction of factors increasing and diminishing stresses / e.g. the influence exerted by sheetings and diaphragm walls on the distribution of stresses, the effect of pressure level fluctuations of deeper lying artesian waters and of pit dewatering, etc./.

Not before we are in possession of knowledge as indicated above shall we be in a position to draw the limits within which conventional methods of calculating settlements can meet the requirements of practice, that is beyond which an analysis of the entire process will be required.

+
Investor: National Investment Agency for
Agency for Hydraulic Projects,
OVIBER, Budapest
Designer: Institute for Hydraulic Planning,
VIZITERV, Budapest
The observations are being made by:
Research Institute for Water
Resources Development, VITUKI,
Budapest

SOME PROBLEMS CONCERNING THE EVALUATION OF CHALK AS FOUNDATION FOR THE HYDROTECHNICAL STRUCTURES. A.V.Leonytohev (USSR)

In a number of oases the design and construction of hydrotechnical structures involve certain difficulties associated with the utilization of ohalk deposits, as foundation, the strength and deformation properties of which are far from having been studied sufficiently.

The studies of typical kinds of pure chalk of the Russian platform (with a content of insoluble sediment of less than 5 per cent) made for some full-scale structures have shown its natural incomplete consolidation; and this can be proved by the relation of porosity ratio e of the chalk of natural formation to porosity ratio of remoulded chalk with a moisture content at yield point e_{w_L} which is 0.9 - I.I.

This specific feature of physical state of chalk substantially affects its geotechnical properties.

For example the tests with application of Maslov - Lourie single-dimension shearing apparatuses have disclosed that the shearing resistance of monolithic chalk depends on "density-moisture content" (the method suggested by N.N.Maslov in I94I). The typical graph (as applied to a chalk variety with γ_d =I.38gr/cm^3) is shown in Fig.I and it is represented by a system of flattening curves with the biggest steepness in the area $\sigma < 0.5$ R_o (where R_o - ultimate resistance of chalk to single-axial crushing).

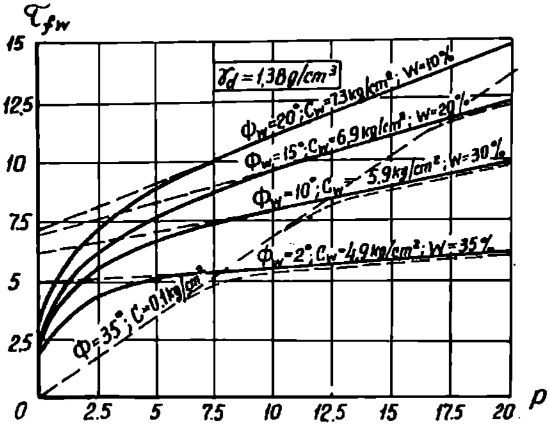

Fig. I. Combined graph of shearing resistance of pure white chalk of undisturbed structure (according to shears of monolithic samples and those with ready made surface of shear)

Notation: continuous line - branches of curve for monolithic chalk; dotted line - branches of curve for shear of chalk along plane of weakness.

The respective analysis confirms regularity of such configuration of the graph branches in the given area σ specified by a possible development of the tensile stresses in the shear zone and mobilization of the chalk structural strength as σ increases in the zones weakened by micro feasures.

At the same time determination of the angle of internal friction φ_w and cohesion C_w by the flat branch of the graph extended to intersection with the ordinate is justified.

Tests have resulted in establishment of relationship between the angle of internal friction and "density-moisture content" (Fig. 2) as well as cohesion which, for chalk varieties with an undisturbed structure and $\gamma_d >$ I.30 gr/cm^3 can be determined from the empirical equation

$$C_w = C_c + \Sigma_w = 63(\gamma_d - 1.30) + 0.1(W_o - W)_2$$

C_c - cement type cohesion kg/cm^2
Σ_w - water film bond kg/cm^2
W_o - full moisture retention capacity in t.
W - moisture content of chalk.

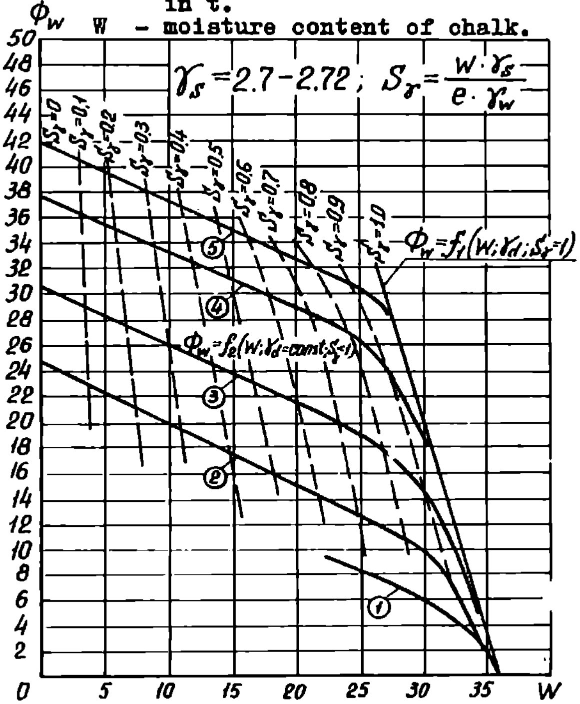

Fig.2. Relation of angle of internal friction φ_w of chalk to moisture content (W) and density (γ_d) of composition

Notation: I - γ_d =I.33g/cm^3; 2 - γ_d =I.38 g/cm^3; 3 - γ_d =I.4Ig/cm^3; 4 - γ_d =I.48 g/cm^3; 5 - γ_d =I.56g/cm^3.

The tests have revealed rather a complicated nature of chalk behavior along the plane of weakness when within the limits of some initial range of normal stresses the shear along the smooth plane is similar to that of two solid bodies at φ = 35° and C = 0.I kg/cm^2. Yet with the increase in a certain load being equal, for example, to 2 kg/cm^2 and 7.5 kg/cm^2 for water saturated chalk with γ_d = I.33 and I.38 gr/cm^3 at W = 42% and 35% respectively the shearing resistance along the plane of weakness is single-valued to that of monolithic chalk. Graphically it is manifested by merging of the moisture one valued branches of the curves showing the shearing resistance of chalk along the plane of weakness and that of monolithic chalk. This can be illustrated by a typical graph I derived for a chalk variety with $\gamma_d = I.38/cm^3$.

One should recognize importance of the established nature of chalk behavior along the plane of weakness in particular for analysis of the structure stability against one dimensional shear.

Of great interest are the studies to determine the coefficient of lateral expension (Poisson's ratio) of chalk and the regularity of manifestation of lateral pressure according to which the coefficient of lateral expansion ν of water saturated chalk at $\sigma < R_o$ which is 0.1 for monolithic chalk reaches the value of $\nu = 0.3$ for microfissured chalk. Thus, in some cases (at $\nu = 0.1$) the settlement of a structure can be determined by the conditions of one-dimensional problem. Three phases of manifestation of lateral pressures of chalk have been established.

The second phase (with loads within the range of $R_o < \sigma < 2R_o$) which is characterized by the coefficient of lateral pressure of $\xi = \frac{\sigma_2}{\sigma_1} = 0.05 - 0.1$ is of an utmost interest from the point of view of designing a foundation.

The analysis of the effect of such a low coefficient of lateral pressure upon the behavior of the foundation and the sequence of the expansion of areas of disturbance (A.V.Leonytchev,1971) has proved the necessity of taking it into account while designing the hydrotechnical structures.

Besides, the studies have shown that the pure chalk is very unlikely to manifest creep deformations which probably will be worth being taken into account in some particular cases; and this can be proved by the coefficient of viscosity of chalk

$$\eta = a \cdot 10^{14} - a \cdot 10^{15} \text{ poises.}$$

The studies have shown that the compression properties of chalk depend on the degree of preservation of structural connections, initial (natural) density and intensity of loads. When load acts upon the kinds of chalk incompletely consolidated by nature their behavior has much in common with that of highly porous britle materials (I.I.Tcherkassov, 1967).

It might be well to point out difficulties that are met in quantitative determination of compression properties of monolithic chalk when pure indicating or weighing methods are used, which lead to underrated values of the modulus of deformation (on the order of $E = 3000 - 5000$ kg/cm^2).

At the same time the deformability of monolithic water saturated chalk with $\gamma_d \geqslant I.38$ gr/cm^2 at loads which do not cause the crushing of its structure is characterized by the modulus of deformation of the order of $E = 10000 - 30000$ kg/cm^2. The deformability was determined by testing prismatic samples with compression strain being measured by tensiometers in the middle part of the samples or by plate loading.

The studies made proved some particular geotechnical properties of chalk depending upon its composition and state, as well as the necessity to study them in detail to make then applicable to a specific hydrotechnical structure to be designed.

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THE INFLUENCE OF LOADING AREA ON SUBSOIL DEFORMATION MODULI AND STRUCTURE SETTLEMENTS.

B.K.Mazurkiewicz /Poland/

During the construction of a heavy dry dock a great number of tests was performed to determine the prospective settlements of the dock for different stages of loadings.The substrata below the foundation level could be divided into three main layers,viz:a)the layer of sandy soils of a mean thickness of about 2.5 m below the foundation base,covering the silty and clayey sands,natural and deposited,the last originating from the exchange of peats and muds;b)the layer of clayey soils covering varved clays of a thickness reaching to about 20m,and c)the layer of gravel soils, about 24m thick.As the most important layer to determine the settlements of the structure,the layer of stiff silty varved clays was taken into consideration, having:unit weight γ=1.90-1.98 G/cm^3,water content w=25,2-28,4%,plastic limit w_P=24.1-24.7% liquid limit w_L=57.7-60.6%,liquidity index I_L=0.14-0.2% and deformation modulus from oedometer tests for stress range up to 1.22 kg/cm^2 E_D=36-49kg/cm^2.

As it was appreciated that the laboratory tests by means of the oedometers have supplied too small values of E_D,it was decided to carry out loading tests covering all basic layers.The loading tests of sandy layers performed by means of square plates of 5000 cm^{2}area and placed about 50cm below the foundation level,have given for the stress range up to 1.25 kg/cm^2 the mean modulus of deformation E_D=181 kg/cm^2.The loading tests of the clay layer were performed with a plate of an area equal to 9.5x19.0=180.5m^2.The modulus of deformation,calculated from the measured settlements was obtained for the stress range up to 1.22 kg/cm^2,equal to E_D=170 kg/cm^2. It has to be noted that the modulus of deformation obtained from the above tests for the gravel layer was equal to E_D=2900 kg/cm^2.

The results of the tests have shown that the assumption of the moduli of deformation, obtained from oedometer tests,in the calculations of the dry dock structure treated as a plate of dimensions in plane equal to 49.7 x 256.7m,would yield too high values of the expected settlements. It would also involve the necessity of performance of very complicated protection structures,particulary on the transition of the crane tracks from the assembly yards to the dry dock superstructure.

Taking into consideration the moduli of deformation obtained from the loading tests, the calculation of settlements of the structure were performed by means of the stress method.It was found that the mean settlement of the sandy layers was equal to s_s=1.83 cm, of the clayey layers s_c=11.35cm and of the gravel layers s_g=1.01cm. The total settlements of the dry dock s = 14.19 cm.

From the beginning of the building period measurements of settlements were performed. As a result it was ascertained that the average settlement of the dry dock during the building period was equal to s=3.54cm,i.e.it very obviated from the calculated value with an assumption of the moduli of deformation obtained from the loading tests.The performed comparisons of the measured settlements: the maximum and average ones,and of the calculated settlements have shown that the real settlements would correspond to the calculated if the modulus of deformation was equal to E_D=340 kg/cm^2.The above corrected modulus was adopted in further calculations and design works.

The measurements of the dry dock settlements in the initial period of dock operation show the existence of elastic deformations of the subsoil,the magnitude of which was depended on the actual state of dry dock loading. Therefore it was decided to perform further measurements of vertical displacements of the dry dock to obtain not only magnitude of elastic settlements but also changes in permanent settlements due to soil interaction on the time-variable loadings.The results of those measurements,performed during the next 6 years of dock operation (1965-1972),have shown, that the difference between the empty and flooded dry dock was 0.6 cm on an average. The maximum differences between the dry dock bottom plate level after disconnecting the installation of the water pressure reduction and the level measured after more than 6 years of further operation were 0.13 and 0.33cm for empty and flooded dry dock respectively. The above results indicate no increase of the permanent settlements of subsoil, although variable loadings were applied. That shows also that the interaction between the structure and subsoil,as a result of certain initial overload of the soil (the building period of the dry dock and the period of action of the installation for water pressure reduction)has caused a soil stabilization sufficient for safe exploitation of the dry dock.

From the performed calculations and measurements the following conclusion can be drawn:

a/ The performed displacement measurements of dry dock with a base of 1276 m^{2}and founded on a varved clay layer show that the modulus of deformation of clay resulting from the measured settlements(E_D=340 kg/cm^2) is not only much greater than the modulus obtained from the laboratory tests(E_D=43 kg/cm^2)but also than obtained from loading tests by means of plate 180.5 m^2 (E_D=170 kg/cm^2).

b/The above statement also shows a considerable interaction between the magnitude of the structure and the reaction of the subsoil-decrease of settlements with the increase of the magnitude of loading area.

c/ The performed displacement measurements of the dry dock during the nearly 10-year period of building and operation have shown that the static loading of the bottom plate to its maximum value and then its stress-relieving allow an application of loadings which change their magnitude without developing additional soil deformations.

RESISTANCE AU CISAILLEMENT AVEC ROTATION DES FONDATIONS MASSIVES.

A.L.Mozhevitinov, S.A.Kouz'mine, A.F.Popov (URSS)

Dans la construction civile et hydrotechnique la perturbation de la stabilité des fondations s'effectue souvent sous la forme du cisaillement avec rotation autour d'un certain pole O. Dans ce cas la capacité portante de la fondation R_τ se trouve inférieure à celle qui a lieu lors du cisaillement de translation R_t. Les auteurs ont élaboré pour la première fois la méthode du calcul de la résistance au cisaillement avec rotation des fondation [1]. Lors de la rotation de la fondation sur la surface de semelle autour du pôle O les contraintes tangentielles réactives limites dans chaque élément dS de la surface de semelle se caractérisent par le critère de Coulomb $\tau = \sigma \operatorname{tg}\varphi' + c'$ et ont la direction opposée au mouvement, suivant la normale vers le rayon-vecteur r de l'élément. La stabilité de la fondation en état limite est déterminée par trois équations d'équilibre:

$$\int_S \tau \sin\psi \, dS = 0; \quad \int_S \tau \cos\psi \, dS = R_\tau; \quad \int_S \tau r \, dS = R_\tau (m+e) \quad (1)$$

où l'origine des coordonnées coincide avec le pôle, l'axe d'où l'on commence à compter l'angle polaire ψ est normal à la force de cisaillement T; e - excentricité de la force T; m, n - projections sur les axes x, y des distances entre le pôle et le centre de gravité C de la surface de semelle pondérée proportionnellement aux contraintes tangentielles τ. La résolution du système (1) définit les coordonnées m, n du pôle, la valeur de la résultante R_τ des contraintes limites τ et le coefficient de sécurité de la fondation $F = \frac{R_\tau}{T}$. L'article [1] donne pour les fondations à semelle rectangulaire les équations analytiques qui découlent du système (1) et permettent de déterminer le coefficient de sécurité pour les cas différents ayant un intérêt pratique. Ces équations correspondent à l'hypothèse de la répartition linéaire des contraintes normales σ selon la surface de la semelle. Les équations théoriques ont été vérifiées par voie experimentale [2]. La capacité portante de la fondation au cas du cisaillement avec rotation a été déterminee sur les plaques rectangulaires rigides de dimensions 1.0 x 0.5 et 1.8 x 0.9 m^2 reposées sur le sable et se trouvant sous les charges correspondant aux nombres de similitude $N_\sigma = \frac{\sigma}{b\gamma} = 0.6 - 2.0$. Les essais ont été realisés pour une large gamme d'excentricités de la force T, ils embrassaient aussi le cisaillement de translation des plaques et leur rotation sous l'action d'un couple des forces. Les résultats de ces investigations ont été généralisés et confrontés avec les résultats des calculs correspondants. La fig. 1 représente les courbes théoriques qui caracterisent la position de pôle $\frac{m}{a}$ et la capacité portante de la fondation $\frac{R_\tau}{R_t}$ en fonction de l'excentricité $\frac{e}{a}$ de la force de cisaillement T. On y a porté aussi les points expérimentaux obtenus pour les plaques de deux dimensions et pour tous les cas de chargement N_σ. Le graphique démontre une petite dispersion des points expérimentaux; en ce qui concerne les valeurs qui les caractérisent elles sont proches aux résultats des calculs théoriques effectués pour une large gamme de charges et d'excentricités.

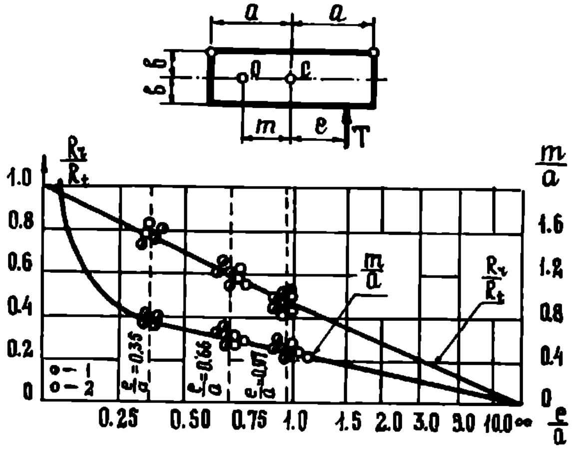

Fig. 1.

1-plauqe 1.0×0.5m^2; 2-plauqe 1.8×0.9m^2.

C'est avec l'augmentation de la charge et de l'excentricité de la force de cisaillement que la divergence entre les résultats expérimentaux et théoriques devient plus accentuée, les résultats expérimentaux donnent une valeur de la capacité portante plus petite que les résultats théoriques, à $N_\sigma = 2.0$ et $\frac{e}{a} \approx 1.0$ la divergence atteint 9% et lors de la rotation de la plaque sous l'action de la couple des forces ($\frac{e}{a} = \infty$) cette diminution devient égale à 12-19%. Evidemment cette divergence est imputable à la non-linéarité des courbes des contraintes normales dans la semelle de fondation. Par exemple, l'emploi des courbes non-linéaires (convexes) des contraintes normales recommandées par VNIIG (Normes II-И 12-67) pour les sables donne une coincidence pratique des résultats de calcul avec les données expérimentales. Donc, bien que dans la plupart des cas les calculs de la résistance au cisaillement avec rotation des fondations puissent etre basés sur l'hypothese de la répartition linéaire des contraintes normales, tout de même dans le cas de la combinaison des charges considérables et de grandes excentricités de la force de cisaillement il faut tenir compte des données plus précises relatives à l'allure de la courbe des contraintes normales.

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AUGMENTATION DE LA RESISTANCE AU CISAILLEMENT ET DIMINUTION DE LA COMPRESSIBILITE D'UN SOL DE MAUVAISE QUALITE POUR FONDATION D'UNE ECLUSE. P.SAVEY et L.GAYET (FRANCE)

Dans le cadre de l'aménagement à buts multiples dit "du Palier d'Arles", la Compagnie Nationale du Rhône a construit une écluse dans le delta du Rhône (France), sur des alluvions récentes non consolidées, formées d'argiles et de silts plus ou moins argileux, et comportant en outre des horizons riches en matières organiques (tourbe).

1 - Le terrain

Le substratum résistant, constitué de sables et de graviers, se trouve à une profondeur de 24 m. Il est surmonté par 16 m de silts argileux et 8 m d'argiles, avec des intercalations de tourbe dont l'épaisseur totale est de 1 m environ.

La nappe phréatique, saumâtre, est située à moins d'un mètre sous le terrain naturel.

Les alluvions récentes (dont les courbes granulométriques moyennes sont représentées sur la figure 1) présentent des caractéristiques mécaniques très médiocres.

Nature du terrain	W %	W_L %	I_p %	φ_u	C_u kPa	φ'	C' kPa	C_c	c_v m²/s
argiles	32 à 60	36 à 73	10 à 41	0	10 à 40	28°	10	0,19 à 0,56	$2{,}5 \times 10^{-7}$
Silts argileux	28 à 49	34 à 41	5 à 15	0	20 à 35	32°	10	0,10 à 0,26	18×10^{-7}

Compte tenu de ces caractéristiques, les difficultés principales étaient les suivantes :

- pour l'exécution des terrassements, le coefficient de sécurité au soulèvement du fond de fouille était inférieur à l'unité, même en rabattant la nappe phréatique (1) ;
- les tassements absolus et différentiels que l'on pouvait prévoir étaient trop élevés pour les ouvrages en béton ;
- sous l'effet du poids des remblais latéraux, il y avait risque de soulèvement du sas de l'écluse lors d'une mise à sec de celle-ci.

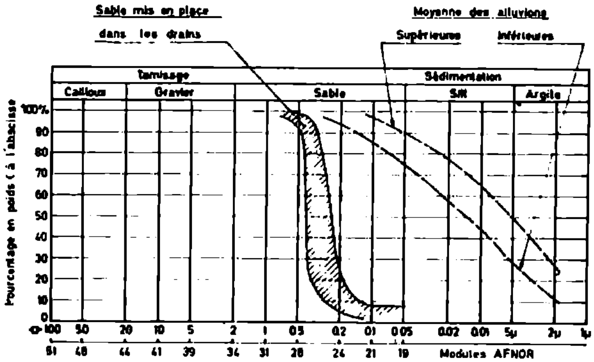

Fig: 1 Courbes granulométriques des alluvions modernes et du sable utilisé pour les drains

2 - Les dimensions de l'ouvrage et de la fouille

L'écluse a 265 m de longueur et 12 m de largeur utile. Compte tenu de l'épaisseur des bajoyers, des estacades, et des besoins de la construction, la superficie du fond de fouille atteint 23 000 m2.

La profondeur de la fouille varie de 6,50 à 8 m sous le terrain naturel. Les remblais latéraux ont une épaisseur de 3 à 5 m au-dessus du terrain naturel,

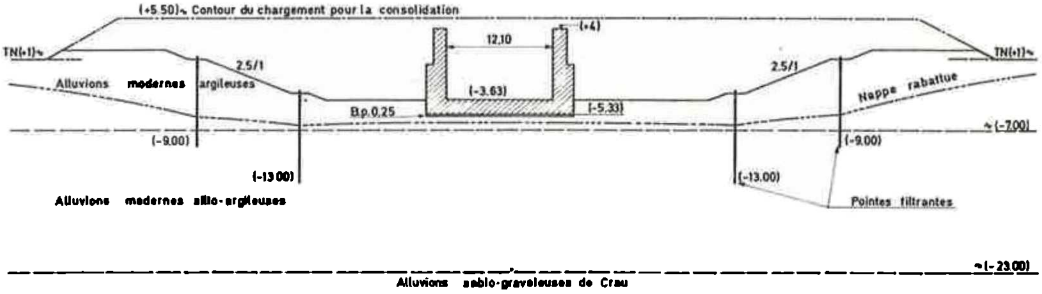

Fig: 2 Coupe transversale de la fouille de l'écluse qui atteint localement 8 m au droit des rampes d'accès du pont franchissant l'écluse.

Le volume total des déblais est de 220 000 m3 ; les bétons représentent 30 000 m3.

3 - La méthode de consolidation

Les études ont montré que la solution la plus sûre et la plus économique consistait à améliorer préalablement la résistance au cisaillement du terrain, en le chargeant par des remblais provisoires, et à fonder les ouvrages sur le terrain ainsi consolidé (2).

Pour mettre au point le projet de consolidation du terrain, il fallait tenir compte des données et des impératifs suivants :

- exécution rapide, de l'ordre de 9 mois, déterminée par le programme général de l'aménagement, dont fait partie l'écluse ;
- utilisation, comme remblais de chargement, des matériaux nécessaires à la constitution des ouvrages définitifs ;
- vitesse de chargement ne dépassant pas 1 m par semaine (risques de poinçonnement du terrain) ;
- anisotropie de la couche argileuse faible $(\frac{kH}{kv} \leqslant 5)$;
- coefficients de sécurité sur les caractéristiques moyennes du terrain après consolidation :
  - pour les ouvrages en terre, 1,5 en cours de chantier et 2 en situation définitive ;
  - pour les ouvrages en béton, respectivement 2 et 3,5 ;
- tassement résiduel ne dépassant pas une dizaine de centimètres pour les ouvrages en béton ;
- recherche d'un degré de consolidation égal à 80 %, pour se limiter à la fraction la plus rapide de la consolidation.

Afin de limiter à 3 mois environ la durée de consolidation après chargement de chaque zone, il a fallu exécuter 4 600 drains verticaux, en sable, espacés de 4 m.

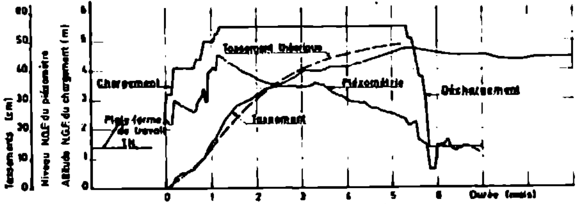

Fig: 3 Consolidation de la zone située au milieu de l'écluse
Mesures des tassements et de la piézomètrie

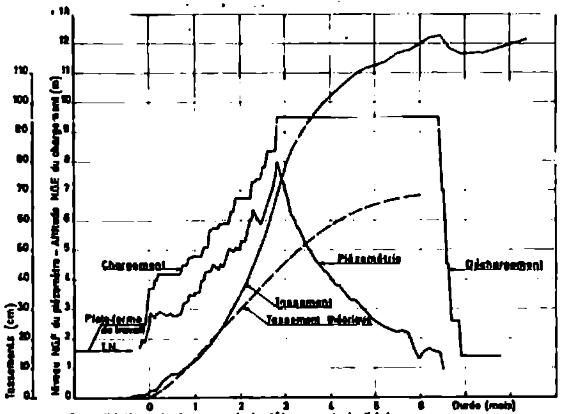

Fig: 4 Consolidation de la zone de la tête amont de l'écluse.
Mesures des tassements et de la piézomètrie

Ces drains, de 0,30 m de diamètre et 23 m de profondeur, ont été réalisés par lançage. Dans les zones soumises à des cisaillements, ils ont été équipés de mèches en fibre de verre. Le fuseau granulométrique du sable utilisé est indiqué sur la figure 1.

Les remblais de chargement, de hauteur comprises entre 4 m et 10 m suivant les zones, avaient une superficie totale de 75 000 m2.

4 - Les résultats obtenus

L'évolution de la consolidation a été suivie au moyen de repères de tassement et de piézomètres.

La durée de consolidation constatée a été conforme aux prévisions.

Les tassements théoriques ont également été atteints pour les zones les moins chargées (4 m de remblais et 0,50 m environ de tassement). Par contre, pour les zones les plus chargées (10 m de remblais), les tassements (1,60 m) ont nettement dépassé les prévisions (0,75 m).

Les figures 3 et 4 donnent l'évolution des tassements et de la piézométrie, au cours de la consolidation, pour deux zones de chargement.

L'importance des tassements absolus et différentiels a confirmé qu'il n'aurait pas été possible de fonder les ouvrages sans traitement préalable du terrain.

Après l'enlèvement des remblais de chargement on a rabattu la nappe à l'aide de pointes filtrantes placées dans des drains de sable situés à la périphérie de la fouille. Les terrassements ont été exécutés sans difficulté jusqu'aux niveaux les plus bas et le bétonnage a été réalisé sans incident.

A l'enlèvement des remblais, on a observé une légère détente du terrain, de l'ordre de 5 à 10 % du tassement de consolidation ; elle a été partiellement éliminée par la mise en oeuvre du béton.

La mise en service de l'écluse a eu lieu en Mars 1973. Les tassements résiduels ont atteint 10 à 15 cm, sans occasionner de désordres aux ouvrages. On peut ainsi considérer que la consolidation a été pleinement efficace.

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(1) La profondeur H de la fouille ne pouvait pas excéder 5,50 m pour avoir un coefficient de sécurité supérieur ou égal à 1. En effet, soit :

- γ = poids spécifique du terrain = 18 kN/m3 (1,8 t/m3)
- Cu = cohésion apparente = 20 kPa (2t/m2)
- Ns = 5 environ
- F = coefficient de sécurité = 1

On a la relation : $\gamma H - \frac{CuNs}{F} = 0$

d'où : $\underline{\underline{H = 5,50 \text{ m}}}$.

(2) Notamment, une solution consistant à reporter les charges sur le substratum graveleux par l'intermédiaire de caissons havés et de pieux, a été étudiée en détail.

GEOTECHNICAL PROPERTIES OF TWO COLLAPSIBLE VOLCANIC SOILS OF LOW BULK DENSITY AT THE SITE OF TWO DAMS IN CANARY ISLAND (SPAIN)

S.Uriel and A.A.Serrano

Among the several types of collapsible soils,this report deals with those which has a sudden change stress of the apparent modulus of deformation when the effective stresses reach a certain level,due to the failure of the bonds between the particles of the soil. This is the case of cemented clays or silts, rocks of a great porosity,volcanic ashes, etc., which collapses when summited to increasing stress field.

In the figure 1 are indicated in a theoretical p-q diagram the conditions which produce the collapse of one soil of this type. The collapse can be originated by the failure of the bonds in three different ways: by tensile,shear and compression stress. Failure by shearing stress can take place if the bonds are short and wide. On the contrary if the bonds are long and slender only collapse by tension or compression is possible. The formulas indicated in fig.1 refer to homogeneous and isotropic soil. If the cementation presents preferential orientations,the relative position of the lines p-q would be different it being possible that one of the three types of failure can predominate.

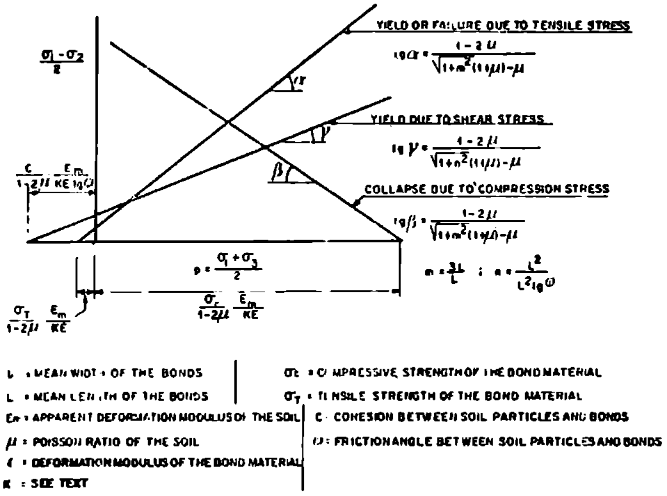

FIG. 1

In the fig.2 and 3 are presented the result obtained in triaxial tests for two volcanic cemented agglomerates of the Canary Island, which will be the foundation soil of two dams Los Campitos and Arinez dams. It can be easily seen the tensile and compresive type of failure,although the tests are few for clearly observed, if it exists,the shear stress failure.

The volcanic soils of the Los Campitos dam,having a bulk density between 0,7 and 1.0, seem to follow a law similar to the one just indicated, although there are not enough data to establish correlation between the different types of failure.

For the volcanic conglomerate of the Arinez dam, which has a higher density (1.2 to 1.4 Tons/m^3, collapse takes place when the major principal stress reaches a value ranging from 27 to 35 kg/cm^2, due probably to an anisotropic distribution of the bonds.

For the Los Campitos dam the solution finally adopted was the removal of the material; For the Arinez dam the major principal stress was required not to exceed 5 kg/cm^2.

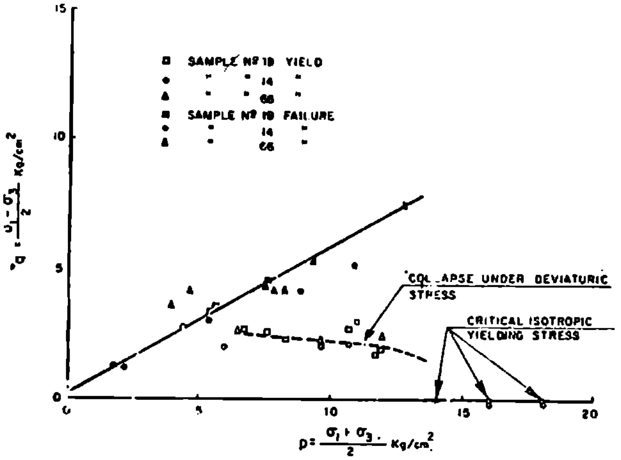

FIG.2.- p-q DIAGRAM FOR q_c AND q_f LOS CAMPITOS DAM

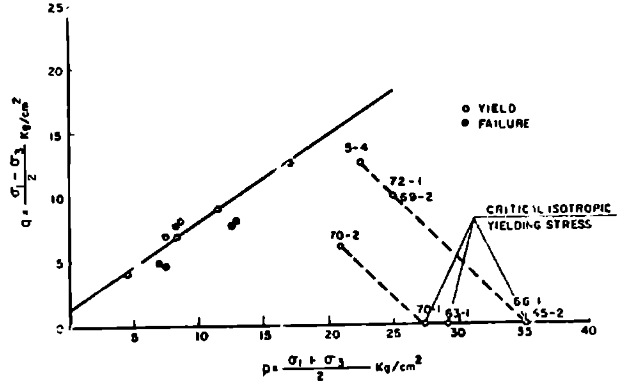

FIG.3 .- ARINEZ DAM p-q DIAGRAM FOR q_c AND q_f