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The Properties of Moraines

Les Propriétés des Moraines

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Summary

This paper gives the results of investigations dealing with the following properties of moraines:

(1) Consistency. The liquid limit and the shrinkage limit of different moraines were determined. It was found that simple relations exist between the consistency limits.

(2) Permeability. The variations of permeability with void ratio and method of compaction were investigated. The test results served as a basis for deducing general relations between void ratio and permeability.

(3) Compressibility and consolidation. Tests were made on moraines differing in clay content. The effects of this soil constituent on compressibility and consolidation are discussed.

(4) Pore pressure and shear strength. Results of undrained shear tests on moraines differing in clay content are described.

Sommaire

Ce rapport présente les résultats de quelques recherches portant sur les propriétés suivantes des moraines:

(1) Consistance. On a déterminé la limite de liquidité et la limite de retrait de plusieurs moraines. On a constaté qu'il existe des relations simples entre les limites de consistance.

(2) Perméabilité. On a étudié les variations de la perméabilité en fonction de l'indice de vide et du procédé de compactage. En se basant sur les résultats des essais, on a établi des relations générales entre l'indice de vide et la perméabilité.

(3) Compressibilité et consolidation. Des essais ont été effectués sur des moraines dont les teneurs en argile étaient différentes. L'auteur discute les effets produits par cette partie constituante du sol sur la compressibilité et la consolidation.

(4) Pression de l'eau interstitielle et résistance au cisaillement. On décrit les résultats des essais de cisaillement à teneur en eau constante faits sur des moraines à différentes teneurs en argile.

Introduction

Moraine soils are the most common deposit in Scandinavia and they also occur in many other parts of the world. During the past few years these soils have been used in Sweden to an ever increasing extent as a material for building earth dams. Because of the special weather conditions, particularly in northern Sweden, the use of the normal roll-fill method has involved certain drawbacks; therefore this method has been replaced by the wet-fill method, which was introduced by the Swedish State Power Board in the construction of the dams at Borga and Ligga (LÖFQUIST, 1953).

The increased use of the wet-fill method is based on the broadening knowledge of the properties of moraines. In soil mechanics these soils are often classified as belonging to a single soil group with similar geotechnical properties. There are, however, many different types of moraine with widely varying properties. Researches into the moraine soils have shown that simple approximate relations can in many cases be established between their properties and certain characteristics of the finest soil constituents, such as the grading and the clay content.

This paper deals primarily with the following properties:

Consistency
Permeability
Compressibility and consolidation
Pore pressure and shear strength

Consistency

The consistency properties of moraines are dependent on the water content, the grain size distribution, and the properties of the fine-grained fraction. In soil mechanics the Atterberg tests are widely used as an aid in the identification and classification of cohesive soils. For most moraines, however, the ranges between Atterberg limits determined in accordance with ASTM standard tests differ little because of the low plasticity.

Fig. 1 shows the relation between the maximum grain size of sieved material and the corresponding values of the liquid limit for two typical moraines and a clayey silt. In this graph the shaded areas include samples from the same geological region. According to Fig. 1 the liquid limit of the clayey silt is independent of the grain size. On the other hand, the liquid limit of the moraines decreases with increasing grain size of the material under test. For each sample, the relation between the liquid limit and the grain size is represented by a curve which approaches a vertical tangent as the grain size increases. The corresponding value of the liquid limit is more representative of the soil than a value obtained by means of the standard test, where only material passing the No. 40 sieve is used.

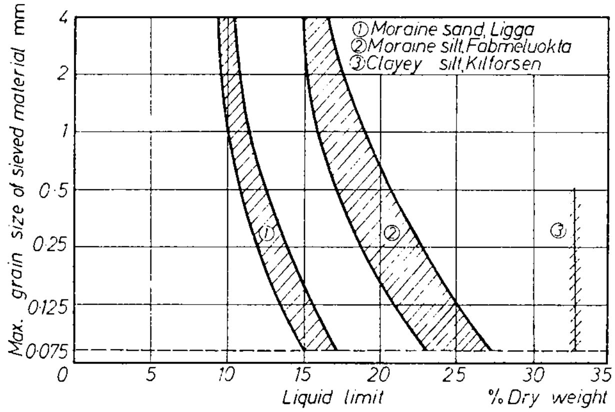

Fig. 1 Relation between liquid limit and maximum grain size of typical moraines and a clayey silt

Relation entre la limite de liquidité et le diamètre maximum des grains des moraines typiques et d'un limon argileux

According to Fig. 1, the liquid limit of a moraine is defined as the tangent value of the liquid limit curve. Thus this value can be estimated by making Atterberg tests on some fractions, obtained by means of mechanical analysis.

Because of the low plasticity of most moraines their plasticity index amounts to only a few per cent if the tests are made according to the ASTM standard. Therefore, in problems met with in the classification of different moraines it is found more convenient to use only the finest constituents when testing the soils. The maximum grain size in such samples is generally fixed at 0·075 mm.

In earlier investigations Swedish geologists have extended the plastic range to the shrinkage limit since they considered the shrinkage limit to be a more important soil characteristic than the plastic limit (Ekström, 1927). This result is also confirmed by investigations of different moraines. As it is difficult to determine their plastic limits, the moraine types are generally best classified according to the liquid and shrinkage limits.

The range of water content between the liquid and shrinkage limits is called the '*LS* difference'. In contradistinction to the plastic index this difference can be estimated for all moraines.

Results from tests made on a great number of different moraines have shown that the liquid limit *LL* increases linearly with increasing *LS* difference, see Fig. 2. This diagram also shows that points representing different samples from the same soil stratum define straight lines which are located in a narrow range close to a straight line *M* represented by the equation

$$LS = (0{\cdot}5 \times LL) - 5 \qquad \ldots . (1)$$

where *LS* is the *LS* difference.

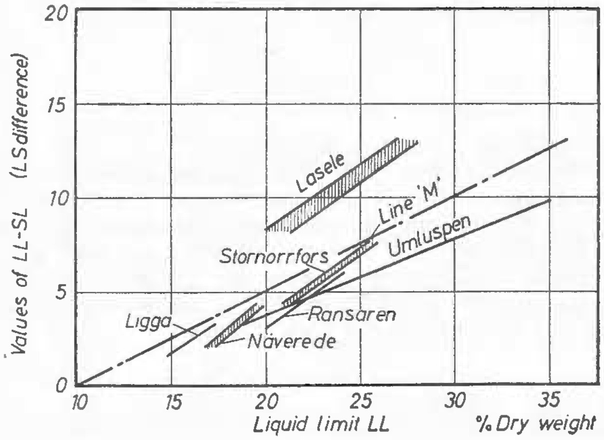

Fig. 2 Relation between liquid limit and values of *LL–SL* (*LS* difference) for moraines

Relation entre la limite de liquidité et les valeurs de *LL–SL* (différence *LS*) pour moraines

The line *M* as defined above characterizes the location of different moraines with regard to their physical properties; therefore in problems concerning the classification of soils this line can be used as a supplement to the *A* line, known from investigations of silts and clays.

Permeability

From earlier investigations it is known that the permeability of soils depends upon several factors. In practice the most important of them are the grain size distribution, the water content at compaction, and the ultimate value of the void ratio which is reached after consolidation.

Experience has shown that for the moraines there are often great difficulties in determining the correct values of the permeability. The causes are generally to be found in variations of the factors mentioned above.

The variations in the permeability due to changes in water content and void ratio are of special interest in the construction of earth dams. The use of the wet-filled method causes high pore pressures, and it is therefore found convenient to choose relatively permeable moraines as core fill material. However, the use of such materials must always be based on laboratory investigations of the permeability limits.

Fig. 3 shows the results of a number of permeability tests carried out on a typical moraine. The investigations included three test series, in which the water content varied from the Proctor optimum value, 6 per cent, to the full saturation value 12 per cent. In each test series the samples were compacted at the same water content to a varying density. The permeability was determined after consolidation of the samples at a normal pressure of 2 kg/cm^2.

As is shown in Fig. 3, the permeability decreases with increasing water content at compaction when the water content is above the value which corresponds to the Proctor optimum. The lowest permeability values are obtained for samples compacted at a water content that is higher than the Proctor optimum value. In a semi-logarithmic plot the relation between water content, void ratio and permeability is represented by the set of curves shown in Fig. 3. At low values of the void ratio the curves approach a straight line (Taylor, 1948).

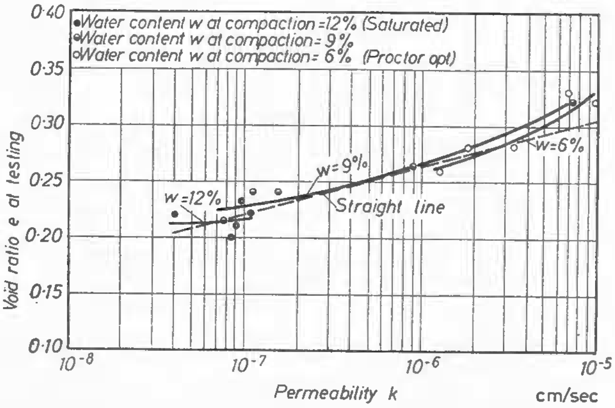

Fig. 3 Relation between water content at compaction and permeability of sandy moraines

Relation entre la teneur en eau au compactage et la perméabilité d'une moraine sableuse

Investigations indicate that the linear relationship is approximately valid for all moraines. Thus, in this straight line range, the permeability for a certain moraine type may be represented by the equation

$$e = e_o + C_p \log_{10} \frac{k}{k_o} \qquad \ldots . (2)$$

where k_o is the permeability at the void ratio e_o.

The slope of the permeability line in a semi-logarithmic plot is designated by C_p, and is called the permeability index. Its value depends on the grain size distribution, the structure of the soil skeleton, and the viscosity of the water.

Laboratory tests on different soils have shown that the permeability index generally decreases with increasing value of the uniformity coefficient D_{60}/D_{10}, see Fig. 4. For sediments, the permeability index is higher than 0·07, whereas for typical moraines its value is approximately constant, about 0·05.

As illustrated in Fig. 3, the permeability of a moraine can vary in several logarithmic cycles according to the water content at compaction. This variation is more prominent for clayey moraines. In standard tests, however, the samples are usually saturated at compaction, and such tests therefore give fairly well defined values, which are located near the lowest part of the permeability line.

Fig. 5 shows the location of different soils according to their permeability. The tests were made on saturated samples consolidated at a normal pressure of 2 kg/cm^2. The assortment, about 2000 samples, included moraines as well as sediments. As is shown in Fig. 5, the moraines are located in a narrow range close to the curve M, extending from $k = 6 \times 10^{-9}$ to $k = 6 \times 10^{-5}$ cm/sec. For each moraine type the relation between permeability and void ratio in the semi-logarithmic plot, i.e. the curve M, is approximately a straight line represented by the equation

$$e = e_o + C'_p.\log_{10}\frac{k}{k_o} \qquad \ldots . \ (3)$$

where k_o is the permeability at the void ratio e_o. The permeability index C'_p in equation 3 varies only in a small range—from 0·01 for clayey moraines to 0·04 for sandy and gravelly soils.

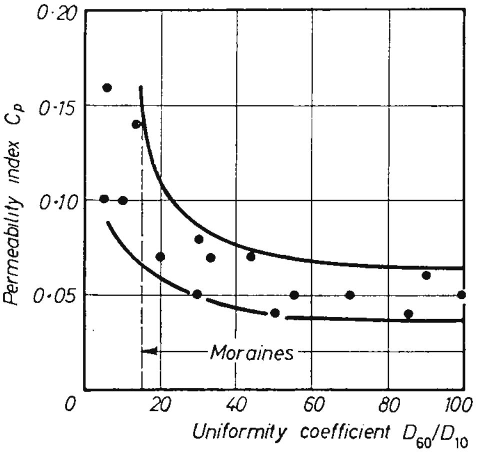

Fig. 4 Relation between permeability index C_p and uniformity coefficient D_{60}/D_{10}

Relation entre l'indice de perméabilité C_p et le coefficient d'uniformité D_{60}/D_{10}

So far experience suggests that the upper limit of permeability in earth dams with moraine fill should not be higher than about 3×10^{-5} cm/sec. However, in gravelly moraines great care must be taken because in these soils there is a limit, determined by the content of fine material, beyond which the permeability is no longer constant in the event of repeated variations in the hydraulic gradient. If the fine material content is too small in relation to the content of heavier grain size, erosion of the fine material can take place, with a consequent rapid increase in the permeability. Experiments have shown that the risk of erosion may be expected not to arise if the content of fine material, less than 0·075 mm, is at least 15 per cent of the material less than 5·6 mm.

Compressibility and Consolidation

Investigations of moraines have shown that the compressibility and the consolidation are principally determined by the clay content. The effect of this soil constituent has been studied in one-dimensional tests made on samples varying in clay content. In the tests the clay content has been determined in per cent of all material less than 5·6 mm, including all material less than 6μ.*

For remoulded samples it is found that the compression curve in a semi-logarithmic plot—in accordance with many clays—is fairly straight over a wide range of pressure $p_o + \Delta p_1$, if the pressure p_o exceeds about 1 kg/cm^2. The slope of this curve is dependent on a soil characteristic, the compression index C_c, which is defined by the ratio

$$C_c = -\frac{\Delta e}{\log_{10}\dfrac{p_o + \Delta p_1}{p_o}}$$

where Δe is the change in void ratio.

From investigations of moraines it is found that the value of C_c increases linearly with the clay content f, as is shown in Fig. 6. For different moraine types, the values of C_c are located within a narrow range, indicated by the dash lines in the graph. The relation between clay content f and compression index C_c can approximately be expressed by the line C_c in Fig. 6. The equation of this line is

$$C_c = 0{\cdot}0044f + 0{\cdot}003 \qquad \ldots . \ (4)$$

For moraines with very low clay content—less than 1 – 2 per cent—the results indicate that the compression index C_c is approximately constant = 0·01.

From investigations of clays it is known that the time required to obtain a certain degree of consolidation is dependent on the compressibility and the permeability. These two properties are usually included in a soil characteristic, the coefficient of consolidation c_v. The value of c_v generally increases with increasing plasticity (Terzaghi). For moraines

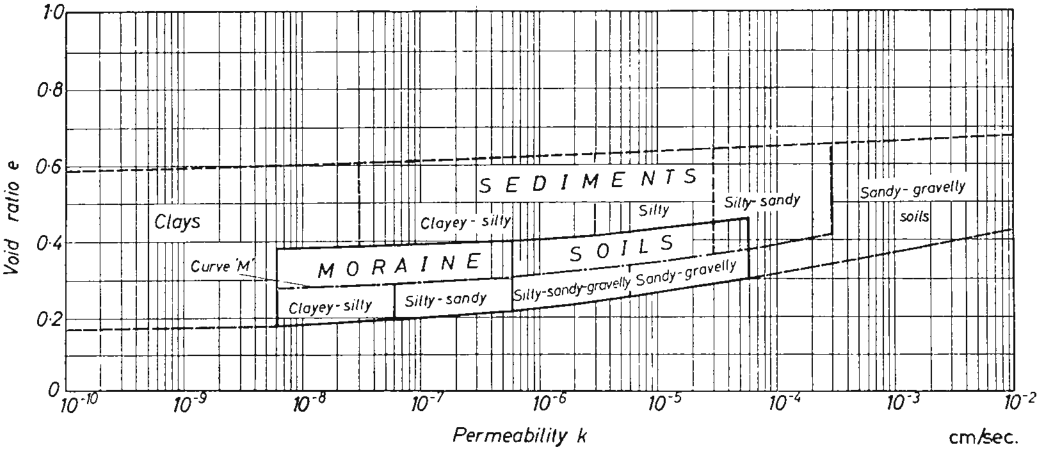

Fig. 5 Relation between void ratio and permeability of different soils
Relation entre l'indice de vide et la perméabilité des différents sols

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* According to the Swedish standard the upper limit for the grain size of clay is fixed at 2μ. However, the very fine silt with grain sizes between 2μ and 6μ shows to a certain degree the characteristic properties of clay and therefore in this paper the clay content f includes all material less than 6μ.

it is found more convenient to bring the coefficient of consolidation into relation with the clay content because of its great importance in the consolidation process.

Fig. 7 shows the results of one-dimensional consolidation tests made on moraine samples from three different geological areas. As is illustrated in this figure, the values of c_v for samples of the same origin are located close to a straight line in a semi-logarithmic plot.

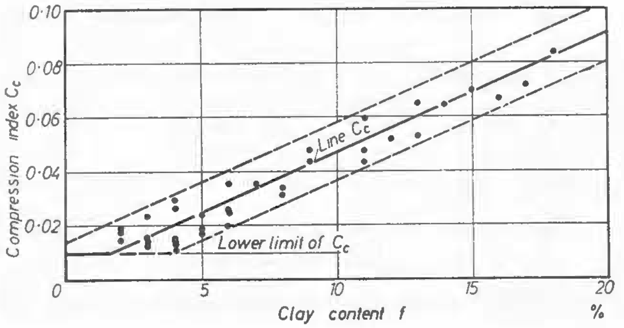

Fig. 6 Relation between clay content and compression index of moraines

Relation entre la teneur en argile et l'indice de compressibilité des moraines

Investigations of different types of moraine show that the straight lines which express the relation between clay content and c_v value have approximately the same slope. All these lines are located within a narrow range between the dash lines shown in Fig. 7.

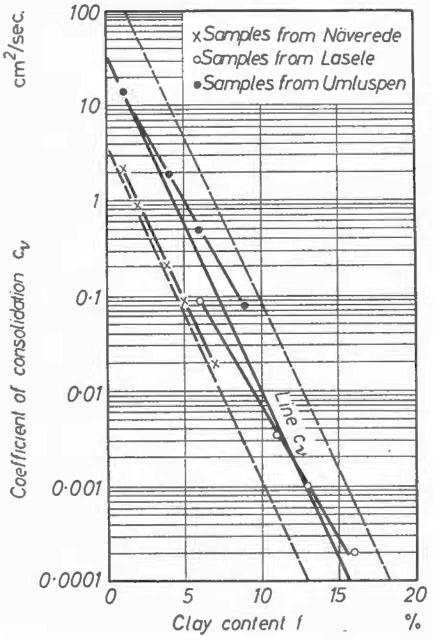

Fig. 7 Relation between clay content and coefficient of consolidation of moraines

Relation entre la teneur en argile et le coefficient de consolidation des moraines

For a moraine, the relation between clay content f and coefficient of consolidation c_v can approximately be expressed by the line c_v in Fig. 7. The equation of this line is

$$\log_{10} c_v = 1{\cdot}50 - 0{\cdot}35f \quad \ldots. \ (5)$$

According to Terzaghi's theory of consolidation, the degree of consolidation after the time t for an open layer with the thickness $2d$ is determined by the equation

$$t = \frac{T_v}{c_v} d^2 \quad \ldots. \ (6)$$

where T_v is the time factor.

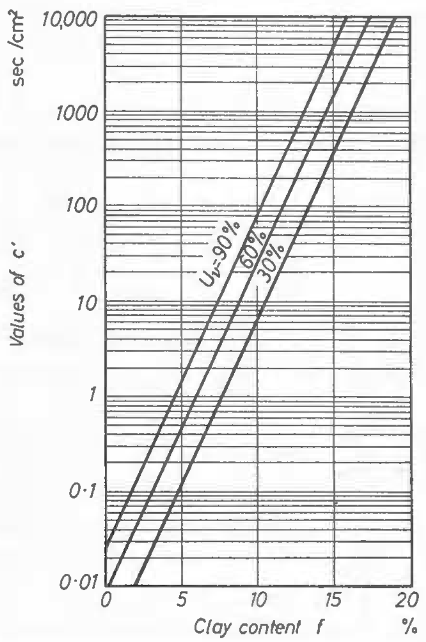

Fig. 8 Relation between clay content and degree of consolidation

Relation entre la teneur en argile et le degré de consolidation

By combining equations 5 and 6 the following relation is obtained

$$t = c'd^2 \quad \ldots. \ (7)$$

The diagram in Fig. 8 gives the relation between the coefficient c', the degree of consolidation U_v, and the clay content f.

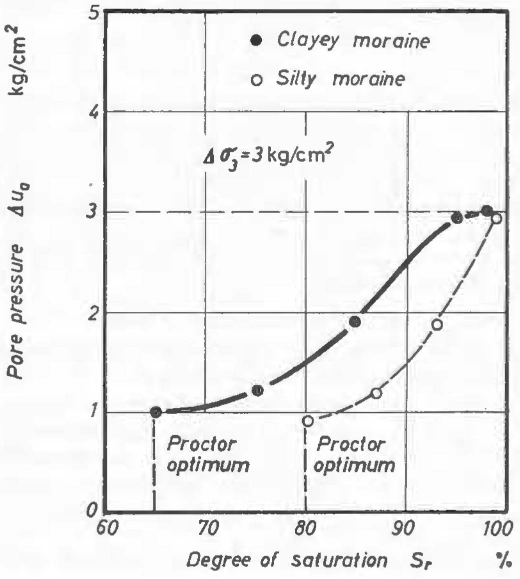

Fig. 9 Undrained triaxial compression tests. Relation between degree of saturation and pore pressure for different moraines

Essais de compression triaxiaux à teneur en eau constante. Relation entre le degré de saturation et la pression de l'eau interstitielle pour différents sols

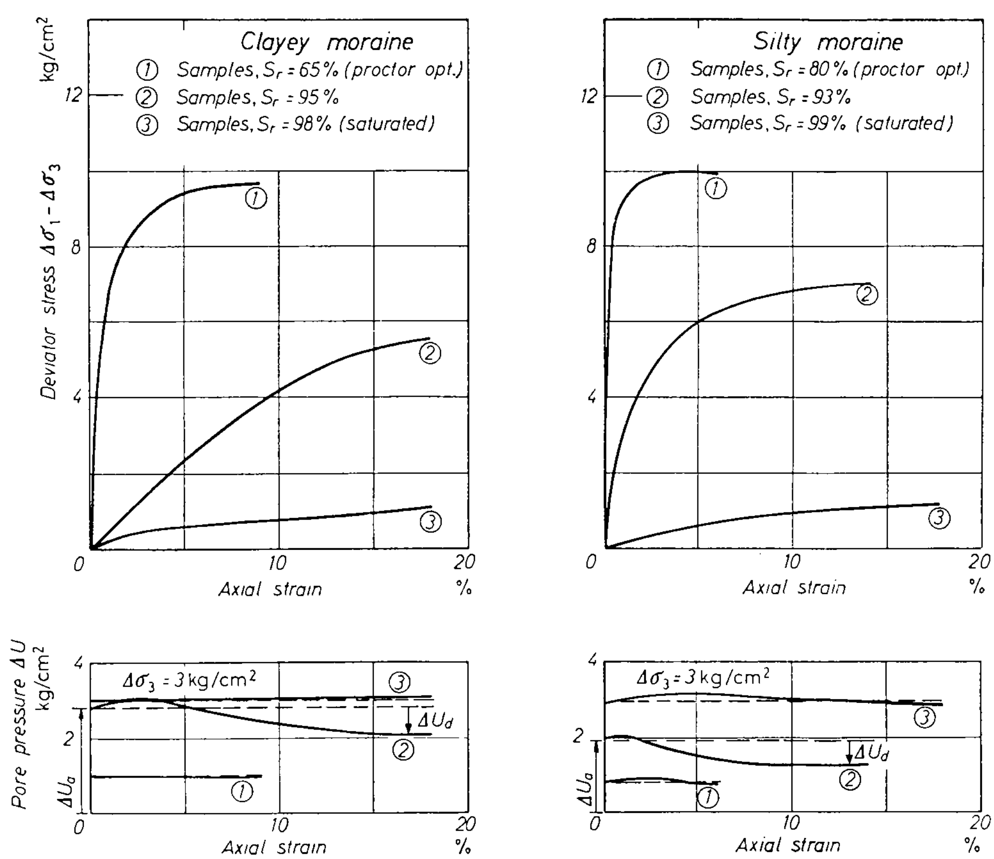

Fig. 10 Undrained triaxial shear tests. Relation between deviator stress, deformation, and pore pressure for partially saturated moraines

Essais de cisaillement triaxiaux à teneur en eau constante. Relation entre l'effort, la déformation et la pression de l'eau interstitielle pour moraines partiellement saturées

Pore Pressures and Shear Strength

The use of the wet-fill method in the construction of earth dams involves several problems of great practical importance. In particular, it is necessary to take into consideration the stability and the deformation of different moraine types during construction and during rapid draw-down.

At the Swedish State Power Board some problems concerning the pore pressures, the shear strength, and the deformation properties of different moraines have been studied in undrained triaxial tests. Two series of tests were carried out on a clayey and a silty moraine with clay contents of 12 and 5 per cent respectively and the results are described in this paragraph. The shear strength of moraines as of clays, is dependent on the pore pressures. These pressures were measured during the tests.

In the tests the coefficients A and B in the pore pressure equation (Skempton, 1954)

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$$

were determined on samples compacted at different water contents to the corresponding Proctor density.

Fig. 9 shows the relation between the degree of saturation S_r and the pore-pressure Δu_a for samples subjected to the same all-round pressure $\Delta\sigma_3 = 3$ kg/cm². As is indicated by the curves in Fig. 9, the difference in pore pressure between the tested moraines is considerable when the degree of saturation is greater than about 80 per cent.

In the cases where the wet-fill method is applied, the degree of saturation is about 90 to 95 per cent. In this range the respective pore pressures Δu_a will be about 90 and 60 per cent of the all-round pressure $\Delta\sigma_3$.

The results of the shear tests are shown in Fig. 10. The change of the pore pressure Δu_d during the application of a deviator stress $\Delta\sigma_1 - \Delta\sigma_3$ is almost the same in the two moraines. At failure the pore pressure Δu_d decreases and this decrease is of about the same magnitude in the two series.

In the table the angle of shearing resistance ϕ, determined at maximum deviator stress in terms of total stresses, is given.

Table

Soil	Test No. 1			Test No. 2			Test No. 3		
	S_r%	ϕ°	ϕ'°	S_r%	ϕ°	ϕ'°	S_r%	ϕ°	ϕ'°
Clayey moraine	65	44	44	95	35	46	98	12	—
Silty moraine	80	45	45	93	40	42	99	12	—

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References

Ekström, G. (1927). *Klassifikation av svenska åkerjordar.* SGU, Ser. C, 345

Löfquist, B. (1953). Earth dams. *Proc. 3rd International Conference on Soil Mechanics and Foundation Engineering.* Discussion

Nilsson, T. and Löfquist, B. (1955). An earth and rockfill dam on stratified soil. The wet-fill method. *5th Congress on Large Dams. Paris 1955. Report No. 32*

Skempton, A. W. (1954). The pore-pressure coefficients A and B. *Géotechnique*, **4**, pt. 4, 143

Taylor, D. W. (1948). *Fundamentals of Soil Mechanics.* New York; Wiley

Terzaghi, K. and Peck, R. B. (1948). *Soil Mechanics in Engineering Practice.* New York; Wiley