7) Distribution of static and dynamic pressure
See Fig. 9, 10 and 11.

8) Field-Tri-axial-Apparatus
See the construction in Fig. 12.

REFERENCES

1) Bendel Engineering-Geology 1948 Vol II p. 200, fig's 209-212.

2) A.R. Anderson. Development and use of electrical extension Meter for the measuring of static and dynamic Fension Swin Archives 1947, p. 321.

3) Bendel. Engineering. Geology 1948, Vo 11 fig 497.

4) For the Derivation of the formula see Bendel Engineering-Geology vol III, 1948 fig 166, 270 270 p. 254 fig. 629.

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SUB-SECTION II e

DIRECT SHEAR TESTS

II e 3

COMPRESSION. AN IMPORTANT FACTOR IN THE SHEARING TEST

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SYNOPSIS

The deformations due to the rotation and the changes of the principal stresses in the shearing test are the cause of the major part of the shearing strain up to relatively high values of the shearing force. The changes to the principal stresses are partly responsible for the "secular effect" in the shearing-strain-time relation at the lower values of the shearing force. Buisman's "secular law" is thus followed, both in the consolidation process and at low values of the shearing force in the shearing process of cohesive, non-consolidated soils.

The mechanics of both processes are analyzed and the results make it possible to compute the strains due to pure shear.

ANALYSIS OF THE SHEARING PROCESS.

The process will be analyzed on the basis of the method of application of normal- and shearing forces, as is usual in slow shearing tests of the constant stress type.

In these tests a vertical load is applied first and sufficient time is allowed for the consolidation of the sample. When lateral deformation of the sample is prevented, a vertical, principal stress p_1, will be accompanied by an "at rest" horizontal, principal stress p_2. This state of stress can be represented by a Mohr's circle (I, fig. 1). The application of a small shearing force will result in a slightly different state of stress, represented by circle II. The transition of state I to state II, because of the application of Δt on the principal plane perpendicular to p_1 can be expressed in a set of formulae:

$$\sigma'_\alpha - \sigma_\alpha = \Delta\tau \sin 2\alpha \qquad (1)$$

and

$$\tau'_\alpha - \tau_\alpha = -\Delta\tau \cos 2\alpha \qquad (2)$$

Where:

$\sigma_\alpha, \sigma'_\alpha$ = the normal stress on an α-plane (see diagram, fig. 1) for state I resp. state II.

$\tau_\alpha, \tau'_\alpha$ = the shearing stress on the α-plane, for the same states of stress.

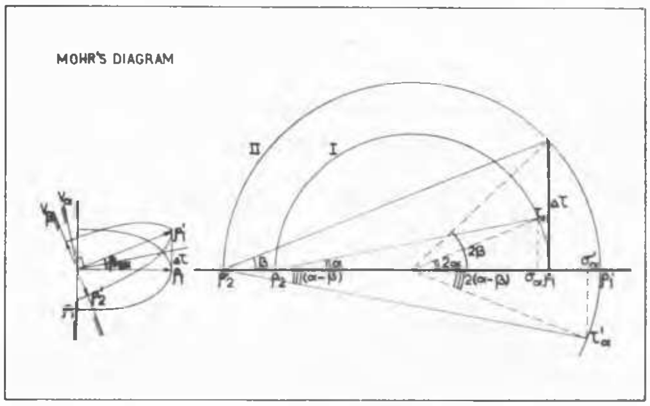

FIG.1

In order to compute the deformations resulting from the changes of the normal and shearing stresses, as indicated by the above-mentioned relations, the layer of a thickness h (see fig. 2) is considered to be composed of prismatic elements of triangular cross-section, with side-planes at an angle of 45° with the vertical plane.

The stresses acting on these planes, for both states of stress, may be computed from:

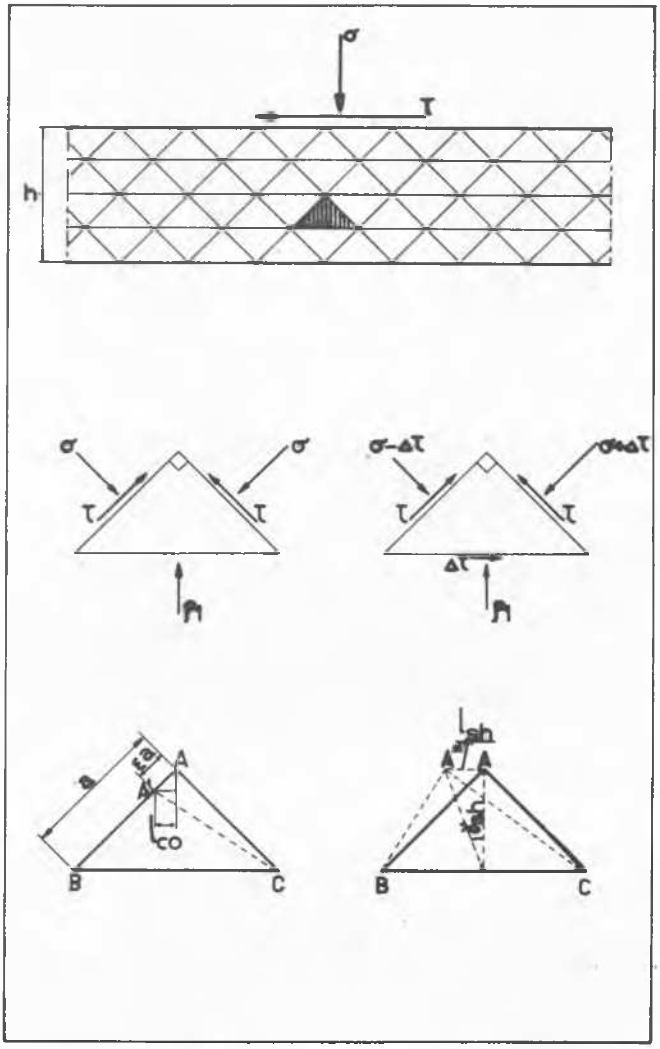

FIG. 2,3,4,5,

State I
$$\sigma_\alpha = a + b\cos 2\alpha \qquad (3)$$
$$\tau_\alpha = b\sin 2\alpha \qquad (4)$$

State II
$$\sigma'_\alpha = c + d\cos 2(\alpha - \beta) \qquad (5)$$
$$\tau'_\alpha = d\sin 2(\alpha - \beta) \qquad (6)$$

Where:
$$a = \frac{p_1 + p_2}{2}, \quad b = \frac{p_1 - p_2}{2}$$
$$c = \frac{p'_1 + p'_2}{2}, \quad d = \frac{p'_1 - p'_2}{2}$$

and β = the angle of rotation of principal stresses, form state I to state II.
From Mohr's diagram (fig. 1) the next relations can be easily derived:
$$\sin 2\beta = \frac{\Delta\tau}{d} \qquad (7)$$
and
$$\cos 2\beta = \frac{b}{d} \qquad (8)$$

In case of $\alpha = 45^0$ resp. 315^0, the values of $\sigma_\alpha, \sigma'_\alpha$ and $\tau_\alpha, \tau'_\alpha$ in (3), (4), (5) and (6) can be computed with the aid of (7) and (8), as:
$$\sigma = \frac{p_1 + p_2}{2}, \quad \tau = \pm\frac{p_1 - p_2}{2}$$
and
$$\sigma' = \frac{p'_1 + p'_2}{2} \pm \Delta\tau, \quad \tau' = \pm\frac{p'_1 - p'_2}{2}$$

As $\frac{p_1 + p_2}{2} = \frac{p'_1 + p'_2}{2}$, the normal stress on the 45^0-planes decreases with $\Delta\tau$, resp. increased with $\Delta\tau$. The shearing stress on the 45^0-planes does not change, from state I to state II (see fig. 3). This result is also obtained with the formulas (1) and (2), which eventually where derived from the formulas (3), (4), (5) and (6).

The above-mentioned relations were also found by Mr. G. de Josselin de Jong 1) graduate civil-engineer, one-time assistant of the Delft Laboratory of Soil Mechanics.

COMPUTATION OF COMPRESSIVE AND SHEARING DEFORMATIONS.

The computation of the deformation of the triangular prisms has to be based on the above-mentioned changes of normal and shearing stresses.

a. The increase of the normal stress from σ to $\sigma + \Delta\tau$ decreases the length of the side AB to A'B (fig. 4).

b. The decrease of the normal stress from σ to $\sigma - \Delta\tau$ increases the length of the side AC. This quantity will be neglected in relation to the preceding one, as the coefficient of compression is small compared with that of decompression.

c. The increase of the shearing stress $\Delta\tau$ acting on BC displaces the top A of the triangle to A" relative to the position of BC (fig.5) Its volume is supposed to remain constant.

a. The decrease of the length AB with time will be computed on the strength of Buisman's "secular law" 2) on consolidation:
$$\varepsilon = \Delta\tau(\alpha_p + \alpha_s \log t) \qquad (9)$$

ε = specific decrease of dimension in the direction of the normal stress $\Delta\tau$

α_p and α_s = constants determined by the consolidation properties of the soil.

With $\varepsilon = \frac{AA'}{AB}$ and AB = a
$$\operatorname{tg}\gamma_{co} = \frac{\Delta l_{co}}{\frac{1}{2}a\sqrt{2}} = \Delta\tau(\alpha_p + \alpha_s \log t)$$

For small values tg γ_{co} is approximately equal to γ_{co}, thus
$$\gamma_{co} = \Delta\tau(\alpha_p + \alpha_s \log t) \qquad (10)$$

which represents the specific shear due to the increase of the normal stress σ to $\sigma + \Delta\tau$.

c. The specific shear resulting from the shearing stress $\Delta\tau$ will in general be a function of $\Delta\tau$ and of time t, so that:
$$\gamma_{sh} = f(\Delta\tau, t) \qquad (11)$$

The sum of both specific shears will be called the total specific shear γ_{tot}. This quantity can be determined by experimental investigation only.

EXPERIMENTAL RESULTS OF SHEARING TESTS ON COHESIVE SOILS.

The relation between the horizontal displacement l_{tot} and the logarithm of time: log t seems to follow Buisman's "secular law" for a number of increases of the shearing force, which had been kept constant each for a relatively long period of time, with the vertical

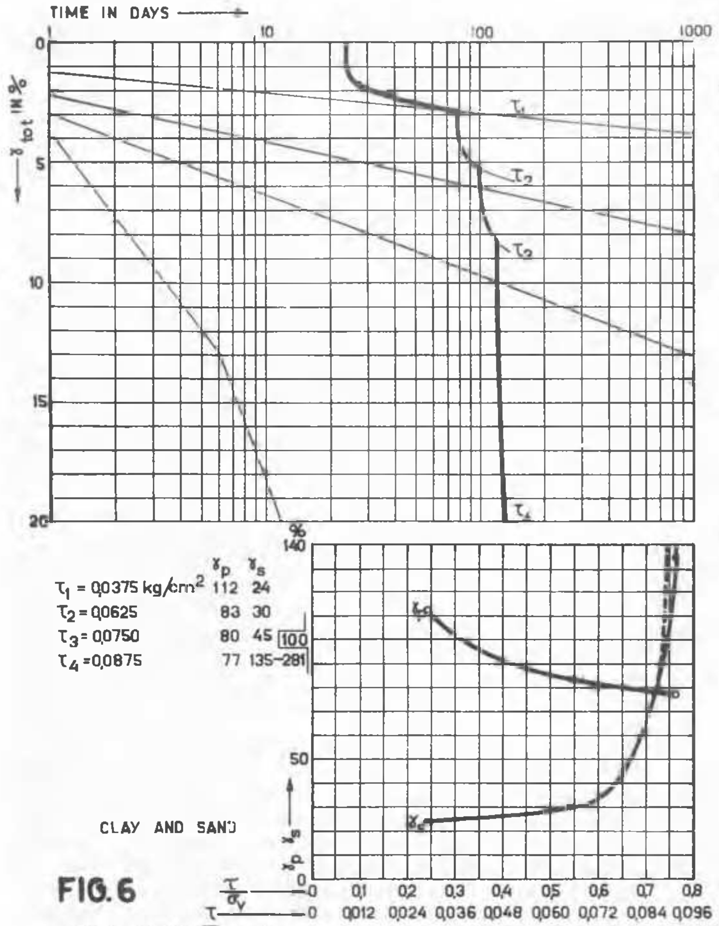

FIG.6

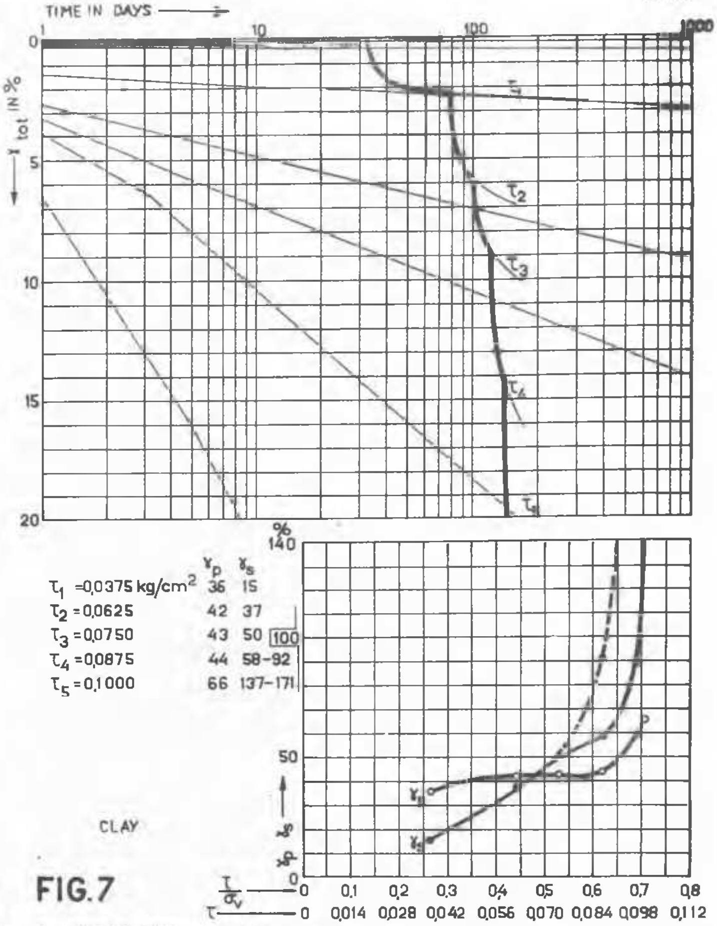

FIG.7

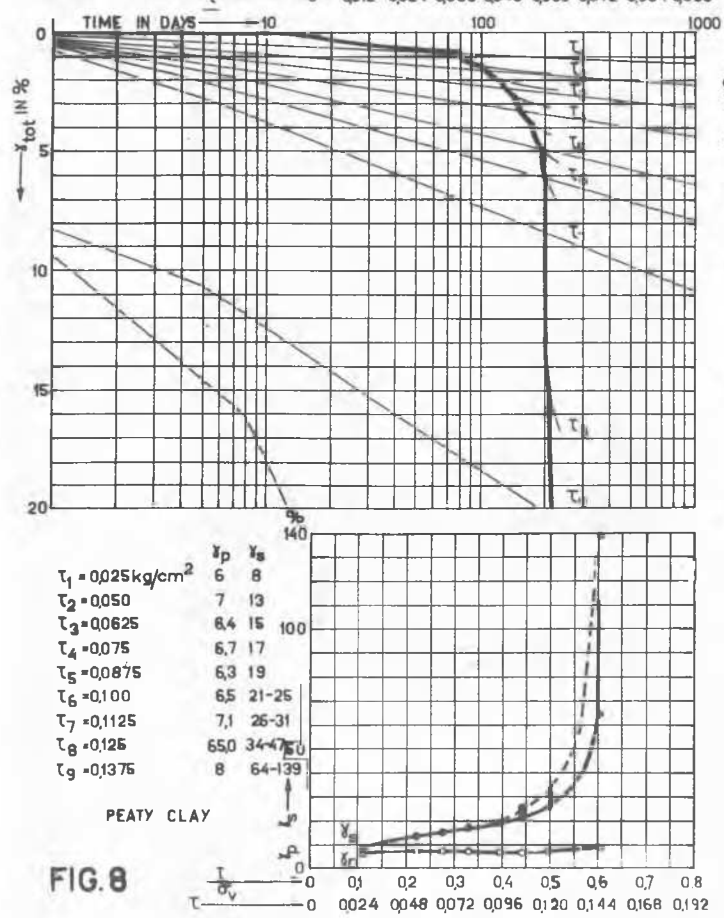

FIG.8

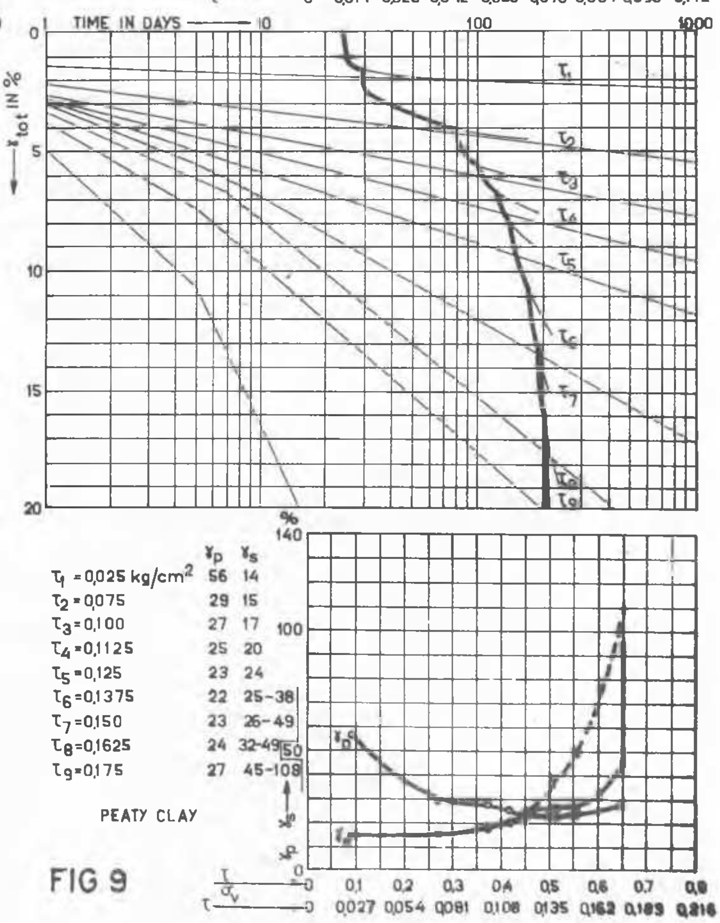

FIG 9

FIG.6,7,8,9

load at a constant value throughout the test.

This fact can be taken from the results of very slow shearing tests on 4 undisturbed samples of cohesive soils, as represented in the diagrams of figs. 6, 7, 8 and 9. The equation of the shearing strain-logarithm of time relation, thus becomes;

$$\gamma_{tot} = \Delta\tau(\gamma_p + \gamma_s \log t) \qquad (12)$$

for the lower values of the shearing stresses, where γ_p and γ_s have a similar meaning as α_p and α_s, replacing the normal stress σ by the shearing stress τ and the specific compression ϵ by the specific shear γ.

On the strength of the reasoning in the preceding paragraph, which has been laid down in the formulas (10) and (11), the value of γ_{tot} in (12) must be equal to $\gamma_{co} + \gamma_{sh}$, thus:

$$\gamma_{tot} = \Delta\tau(\gamma_p + \gamma_s \log t) = \Delta\tau(\alpha_p + \alpha_s \log t) + f(\Delta\tau, t) \qquad (13)$$

From this equation we may conclude, that the specific shear, because of pure shearing stress:

$$\gamma_{sh} = f(\Delta\tau, t) = \Delta\tau\{(\gamma_p - {}_p) + (\gamma_s - \alpha_s)\log t\} \qquad (14)$$

As γ_p and α_p, tespectively γ_s and α_s are of the same order of magnitude, the following conclusions can be drawn from the equations (10), (11) and (14).

1. The application of a relatively small shearing stress $\Delta\tau$ to the horizontal planes of a mass of soil, with initially vertical and horizontal principal stresses, is accompanied by deformations, which are composed of compressive strains and shearing strains.
2. The horizontal components of the compressive strains seem to constitute a considerable part of the total shear, as found in shearing tests on compressive, cohesive soil samples in the lower range of the shearing force-increments.
3. This fact is confirmed in a qualitative sense by the results of shear tests, which show the same fundamental relation of the total shear to the logarithm of time for an increment of the shearing force, as has been found by Buisman for the vertical compression in the consolidation test.
4. This fact has to be confirmed in a quantitive sense by the results of simultaneous shearing and consolidation tests on cohesive soils. Two tentative conclusions may be drawn:
5. In the lower range of the shearing force-increments especially, the total shear as found in tests results will be considerably reduced, when the effect of compression strains is taken into consideration.
6. This fact involves a greater probability of the existence of the so-called "bond stress", as has been defined by Terzaghi. 3) It may be of fundamental importance in considerations of a pure rheological character and in the computation of deformations based on stress-time relations in soil-mechanics problems.

Similar attention has been paid by the author to the effects of shearing stresses on the results of consolidation-tests.

From the point of view of the state of stress, this case is complicated to a large extent by side-friction. This quantity was measured experimentally for a number of different soils. The results lead to the tentative conclusion, that the magnitude of the side-friction may be as high as 20% of the vertical load. Though the results may to some extent be affected by the method of testing, it seems that the shearing stresses thus acting on the circumference of the sample have a fundamental effect on the state of stress.

The mechanics of this process may however be analysed in a similar way as the shearing test, with the exclusion of side-friction. It is then apparent, that shearing stresses will result in a vertical deformation only, the possibility of a radial deformation being excluded.

The evaluation of the vertical deformation in the consolidation process due to shearing stresses will be the subject of future investigations.

Finally the author wishes to express the following view. In the case of all cohesive soils in the state of incomplete consolidation and likewise with all loose-packed non-cohesive soils, a change of the state of stress generally is accompanied by a change of the structure of the soil particles. These changes of structure are the principal source of the deformations. They are thus independent of the type of stress, either normal or shearing. Though from the point of view of the mechanics of materials must be discerned between both types, their effect on the change of the structure is fundamentally the same for a large range of soils.

The secular law on shearing was applied in order to solve a problem connected with the horizontal pressure against a row of piles (see report for this Conference by the same author).

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1) G. de Josseling de Jong: Hypothesis of Hydroforic Deformations, August 1942, Delft Laboratory of Soil Mechanics (not published).
2) A.S. Keverling Buisman: Results of Long Duration Settlement Tests, Proc. Harvard, Conf. Vol. I, 1936, p. 103-106.
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