level and through twin 700 mm discharge pipes to hoppers at a rate of about 8000 cubic metres per hour of diluted mixture with an average density of 1.5 rising at times to 1.6. Whole hopper loads (4000 tons) at mean density of 1.5 have been obtained indicating that less than 40 per cent by volume of dilution water was used (i.e., over 60% of in situ mud in the mixture).

The mixture flows out almost freely from the hoppers when the doors are opened. It is believed that easy pumping of this high density fluid with ordinary mineral constituents is almost unprecedented and indicates that the prolonged grinding processes, involved in the Yangtze silt transport, reduce the sediment to smoothly rounded or flaky grains. These form a mud with low volume of voids, so that when the void volume is only slightly increased the whole mass becomes mobile. The most comparable conditions are those in the Punta Indio at the mouth of the La Plata, but the density of the in situ mud there is only about 1.5, and the dredger builders have not claimed to pump densities exceeding 1.33 (corresponding to 66% in situ material in the mixture).

In the preliminary builder's trials of the dredger made in the approach to Koenigsberg (E. Prussia) the muddy sand there was found to have a bulk density of 1.25 with ordinary mineral constituents. Taking the mineral density as low as 2.5 this corresponds to 84% of voids (porosity coefficient 5.5). Similar voids occur in fairly well packed snow, indicating that a spicular form of grain or honeycomb structure can permit large void volumes.

The range of density and porosity of these three samples all of which behave rather similarly in mud pumps shows how important the investigations of Dr. Terzaghi are in relation to dredging practice.

No. Z-3 STRESS DISTRIBUTION IN DRY AND IN SATURATED SAND ABOVE A YIELDING TRAP-DOOR

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Notations.

s = unit weight of dry or saturated sand
s_o = unit weight of water
φ = angle of internal friction of the sand
$2b$ = width of the trap-door
ℓ = length of the trap-door
H = depth of fill above trap-door
Δh = distance of vertical movement, positive in the downward direction
Q = total pressure of the sand on the trap-door
n_{II} and n_I = normal stress on a horizontal and on a vertical element through a point located above the axis of the trap-door, Fig. 1.
K_o = hydrostatic pressure ratio for lateral confinement (1)
$K = n_{II}/n_I$ = hydrostatic pressure ratio for an arbitrary point located above the axis of the trap-door after the trap-door yielded
$K_r' = \tan^2 (45 + \frac{\varphi}{2})$ = maximum value of Rankine's hydrostatic pressure ratio

Purpose of investigations. A knowledge of the pressure distribution in the sand above a yielding trap-door represents the prerequisite for a clearer understanding of the stress distribution in sand around tunnels. According to the traditional theories of arching above tunnels, including the latest one of A. Caquot (2) the hydrostatic pressure ratio K in the sand located above the roof of a tunnel is equal to the maximum Rankine value K_r'. This assumption appears incompatible with the results of theoretical investigations of the author concerning the process of arching in sands. (3) According to these results the hydrostatic pressure ratio K represents a statically indeterminate quantity with a value intermediate between K_o and K_r'. In order to obtain conclusive information concerning this question, experimental investigations were made by K. Kienzl in the laboratory of the author in Vienna on a trap-door 7.3 cm wide and 46.3 cm long.

Mechanical effect of the yielding of a trap-door. Let us consider a stratum of dry sand, Fig. 1 a, with a unit weight s and a horizontal surface, supported by a rigid horizontal base which contains a rectangular trap-door, aa_1. Before the trap-door changes its original position every part of the base, including the trap-door, is under a vertical pressure which acts like the pressure of a liquid on every part of the base with the same intensity, H_s, per unit of area. However, the slightest downward movement of the trap-door suffices to reduce the vertical pressure onto the door to a small fraction of what it was before. This fact has been known for more than one hundred years.

The mechanics of this transition from the original into the ultimate state of stress are shown in Fig. 1 a and b. As long as the downward movement of the trap-door remains very small it merely produces a vertical expansion and lateral contraction of the lower part of the body of sand, aa_1bb_1, Fig. 1 a, located above the trap-door. As a result of this deformation, the sand located on both sides of this body is allowed to expand laterally, like the backfill of a yielding retaining wall. Since a lateral expansion is always associated with a vertical contraction, shearing stresses develop within two inclined zones, ac and a_1c_1. These shearing stresses transfer part of the weight of the sand located between ac and a_1c_1 onto the undisturbed part of the sand beyond the zone aca_1c_1.

As the trap-door yields still farther, the structure in the sand located immediately above the door disintegrates, owing to excessive expansion, whereupon the planes of minimum resistance shift from the position shown in Fig. 1 a to the boundaries a b and a_1b_1 of the zone of disintegration in Fig. 1 b.

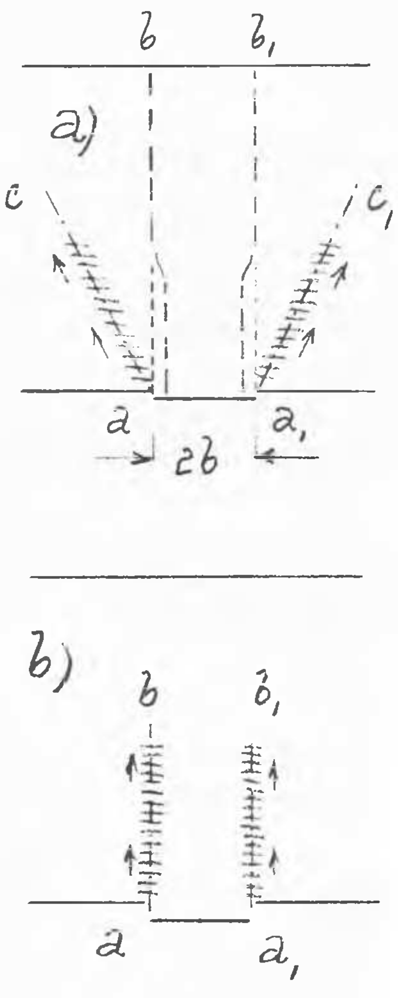

Fig. 1

Tests results. Curve C_1 in Fig. 2 shows the relation between the vertical downward movement, Δh, of the trap-door and the total vertical pressure, Q, on the yielding door for a dense sand, and curve C_2 the same relation for a loose sand. On each curve, the minimum value of the pressure Q corresponds to the state represented by Fig. 1 a and the ultimate value Q_{max} to that shown in Fig. 1 b. The ultimate value for the dense sand is the same as that for the loose sand.

Fig. 3, a to c, shows the distribution of the vertical pressures n_I and the horizontal pressures n_{II} over a plane, vertical section through the axis of the trap-door, (a) for the state preceding the downward movement of the trap-door, (b) for the state corresponding to Q_{min}, and (c) for the state corresponding to Q_{max}. The abscissae of the dotted line represent the ratio between the horizontal and the vertical stress for different elevations above the trap-door. All the diagrams refer to the sand in a compacted state. The stresses n_I and n_{II} were determined by the steel tape method (3). These diagrams confirm the conclusion of the author that the ratio n_{II}/n_I is a statically indeterminate quantity and they also show that the value of this ratio never exceeds about 1.6. The angle φ for the dense sand was equal to 44° and the corresponding Rankine value K_r' equal to 5.6. Fig. 4 shows the relation between the pressure Q and the value Δh for an alternating up- and downward movement of the trap-door beneath a layer of (a) dense and (b) loose sand.

Effect of seepage on the trap-door pressure Q. In order to investigate this effect, a permeable trap-door was used which permitted the maintenance of the stationary flow, within the sand, represented by the flow net shown in Fig. 5. If the flow of the water does not interfere with the stress distribution associated with arching, the pressure on the trap-door for each stage of the test should be equal to the pressure exerted by the dry sand multiplied by a certain ratio s_1/s. In this ratio the value s_1 represents the sum of the unit weight s of the dry sand, the weight of the water contained in the sand and the vertical component of the vertical component of the forces exerted by the flowing water onto the sand per unit of volume of the sand. The flow net Fig. 5 permits the computation of the value of s_1 and, as a consequence, the pressure Q on the trap-door. The results are as follows:

	Q Computed	Q Measured
Dense sand	2.33	2.24
Loose sand	3.82	3.62 kg

Hence the percolation of the water through the sand does not seem to interfere with arching, provided the sand has no opportunity to escape through the openings of the permeable trap-door.

Stability of the stress conditions in the zone of arching. Many engineers still hold the opinion that the state of stress associated with arching is less stable than the classical state of stress assumed by Coulomb and Rankine. This opinion does not seem to be warranted because every temporary change in the stress conditions of a mass of sand has a tendency to relieve the frictional stresses within this mass, and, as a consequence, to increase the pressure on the support. Both the classical states of stress and the state of stress in a zone of arching involve full mobilization of the frictional resistances within the sand. Therefore it is difficult to see why they should be effected by temporary changes in a different way.

The most effective agent tending to reduce friction consists of vibrations. There seems to be no doubt that energetic, continued vibrations are capable of transforming any state of stress in a sand into an almost hydrostatic state involving approximate equality between horizontal and vertical pressure for any point of the material. If such an event could occur under practical conditions, the following results would ensue.

Cement and grain bins would fail, because the computation of the thickness of their walls is based on permanent arching. The side walls of subways and the walls of cellars next to streets with heavy traffic would fail because they are not strong enough to withstand the full hydrostatic pressure of the adjoining soil. The unlined floor of railroad tunnels through sand would gradually work up into the tunnel, because their stability depends on arching in the sand beneath the tunnel.

Since none of these events ever occurred, we are entitled to assume that the vibrations which act on sands under normal conditions merely produce a relatively unimportant increase of the pressure without ever leading to a complete elimination of the frictional stresses. In this connection it makes no

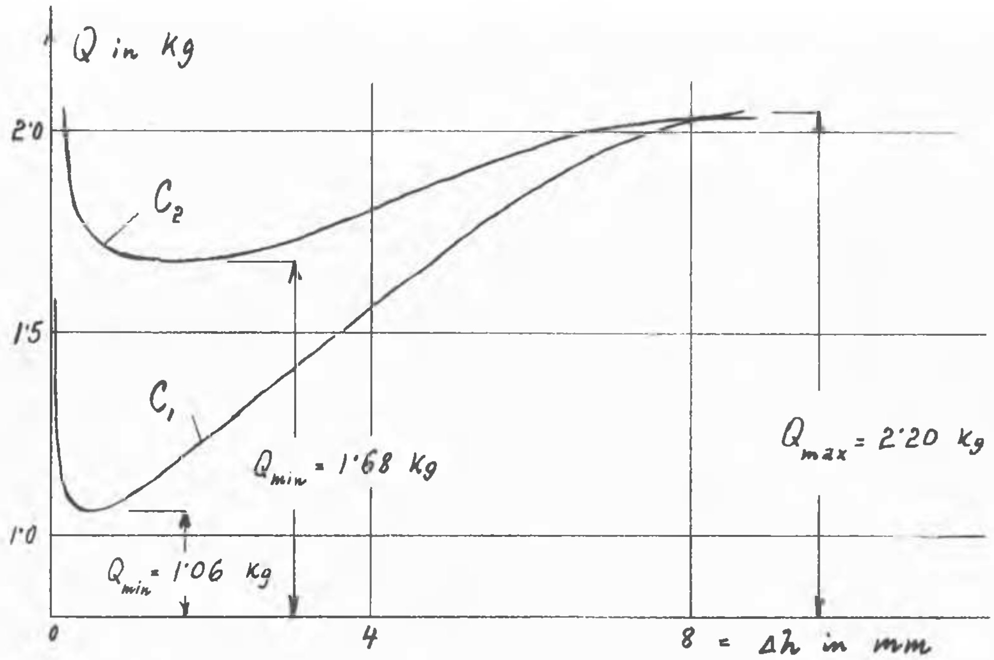
Q in Kg
2·0
1·5
1·0
0
4
8 = Δh in mm
C₂
C₁
Q_min = 1·68 Kg
Q_min = 1·06 Kg
Q_max = 2·20 Kg

FIG. 2

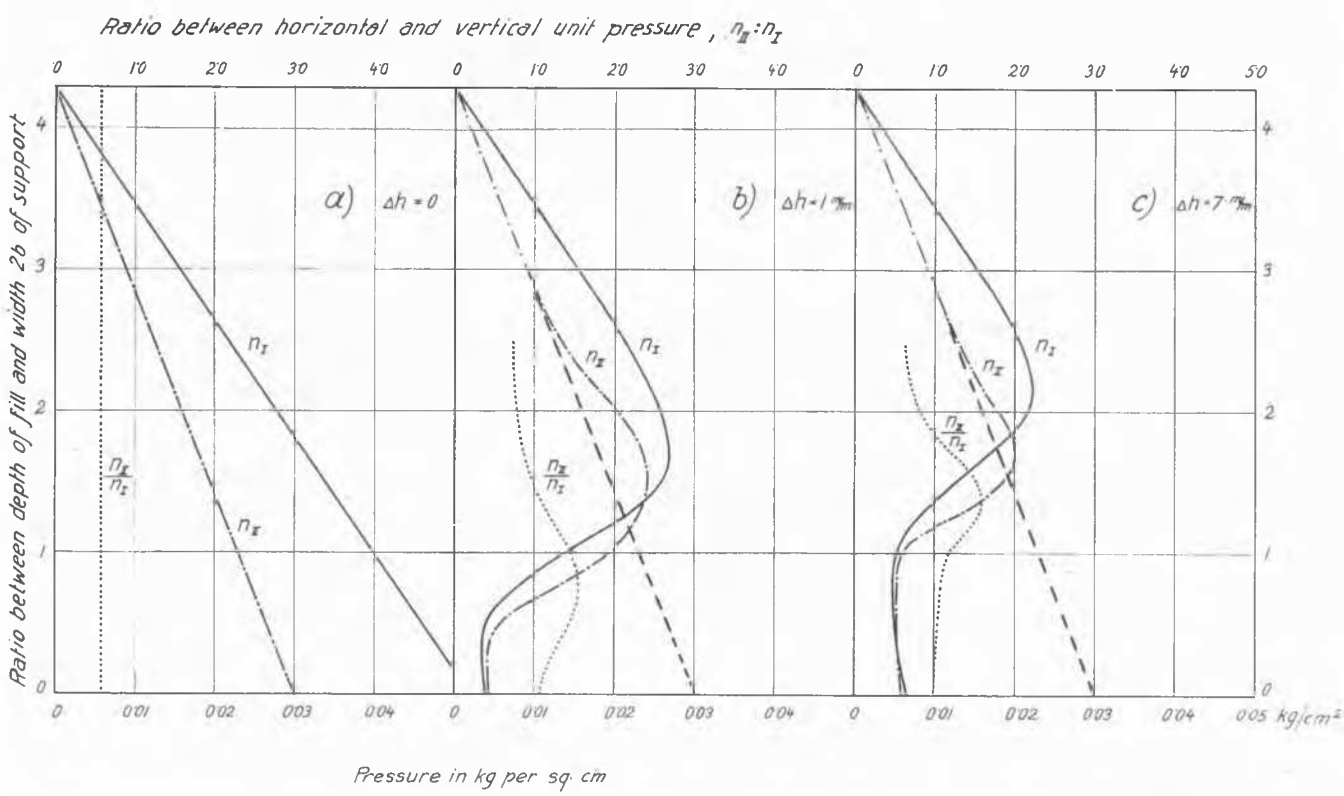
Ratio between horizontal and vertical unit pressure, n_II : n_I
Ratio between depth of fill and width 2b of support
a) Δh = 0
b) Δh = 1 mm
c) Δh = 7 mm
n_I
n_II
n_II / n_I
0·05 kg/cm²
Pressure in kg per sq. cm

FIG. 3

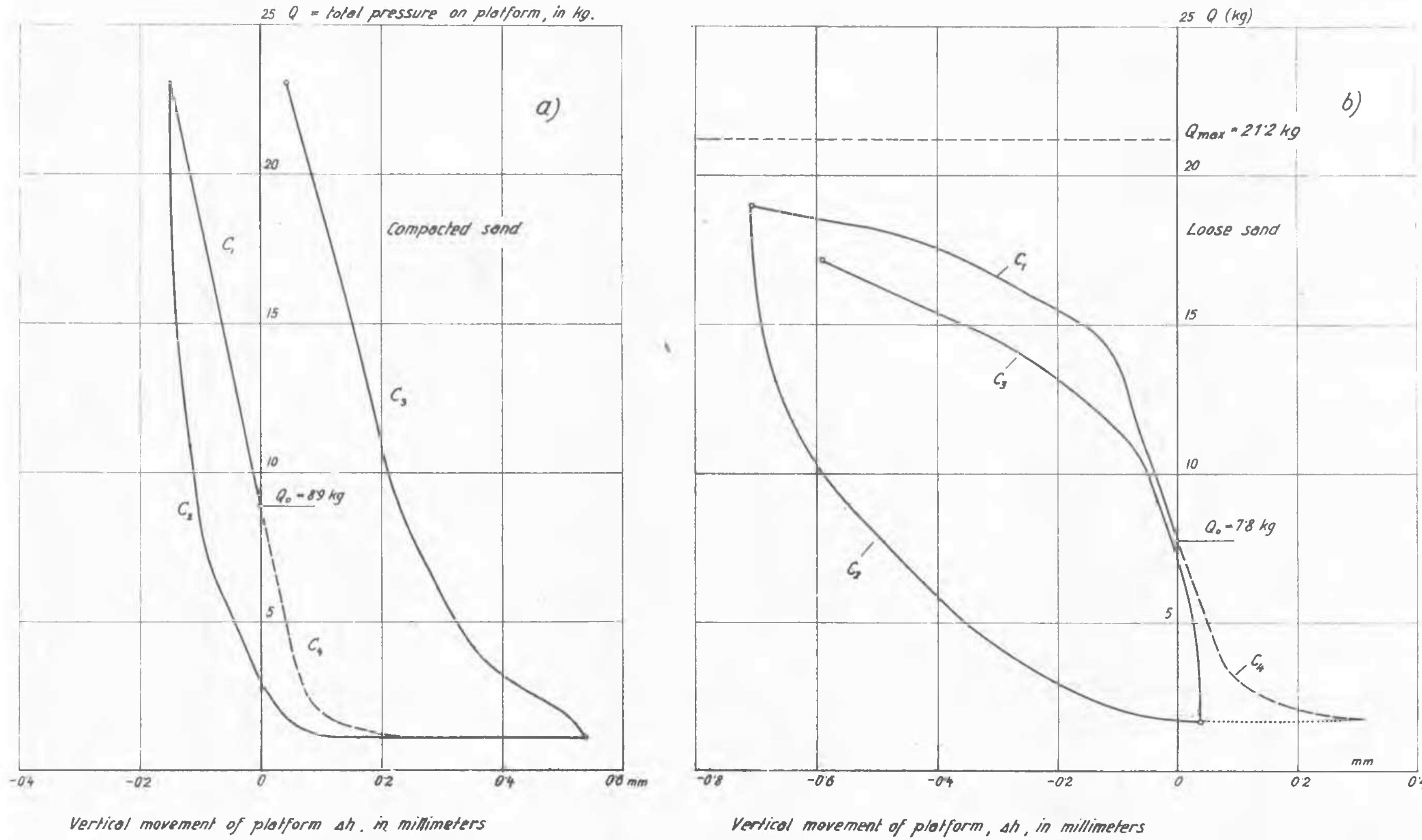
25 Q = total pressure on platform, in kg.
a)
Compacted sand
Q_o = 8·9 kg
C_1
C_2
C_3
C_4
-0·4
-0·2
0
0·2
0·4
0·6 mm
Vertical movement of platform Δh, in millimeters
25 Q (kg)
b)
Q_{max} = 21·2 kg
Loose sand
Q_o = 7·8 kg
-0·8
-0·6
-0·4
-0·2
0
0·2
0·4
mm
Vertical movement of platform, Δh, in millimeters

FIG. 4

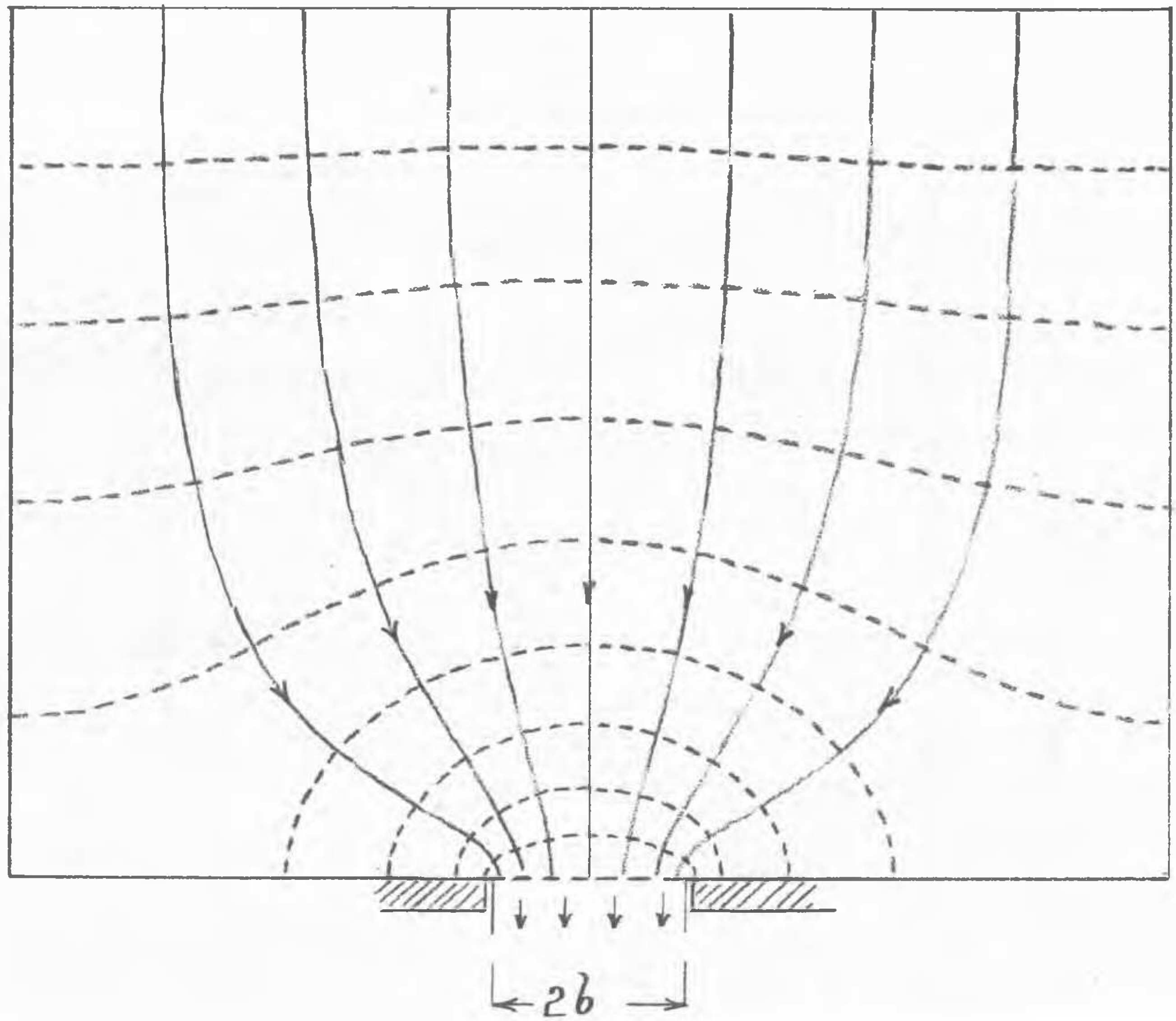

Fig. 5

difference whether these stresses are limited to zones of arching or whether they act as assumed by Coulomb and by Rankine. Conclusive information on the importance of this percentage reduction can only be obtained by direct measurement under field conditions.

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(1) Terzaghi, K., A fundamental fallacy in earth pressure computations. Journal Boston Soc. of C. E., April 1936.
(2) Caquot, A., Equilibre des Massifs a Frottement interne. Paris, 1934.
(3) Terzaghi, K., Erdbaumechanik, Vienna 1925, p. 187.

No. Z-4 INVESTIGATION OF THE BEARING-POWER OF THE SUBSOIL (ESPECIALLY MORAINE) WITH 25 X 25 MM POINTED DRILL WEIGHTED WITH 100 KG WITHOUT SAMPLES

O. Godskesen, Geo-technical Engineer of the Danish State Railways, M.S.D.C.E.

The weighted drill point was introduced in 1917 by Mr. John Olsson of the Geo-technical commission of the Swedish State Railways ("Statens Järnvägars Geotekniska Kommission 1914-22, Slutbetankande, Stockholm 1922" and "Der Bauingenieur 1930 Heft 41".) and has, without appreciable change, since 1927 been the most used tool in the subsoil investigations of the Danish State Railways ("Ingenioren" 1930, No. 44").