KBS TEKNISK RAPPORT

22

The influence of rock movement on the stress/strain situation in tunnels or bore holes with radioactive canisters embedded in a bentonite/quartz buffer mass.

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Högskolan i Luleå 1977-08-22

KÄRN - BRÄNSLE - SÄKERHET

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THE INFLUENCE OF ROCK MOVEMENT ON THE STRESS/
STRAIN SITUATION IN TUNNELS OR BORE HOLES WITH
RADIOACTIVE CONISTERS EMBEDDED IN A BENTONITE/
QUARTYZ BUFFER MASS.

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TEKNISK RAPPORT KBS 5

Report on

THE INFLUENCE OF ROCK MOVEMENT ON THE STRESS/STRAIN SITUATION IN TUNNELS OR BORE HOLES WITH RADIOACTIVE CANISTERS EMBEDDED IN A BENTONITE/QUARTZ BUFFER MASS

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THE INFLUENCE OF ROCK MOVEMENT ON THE STRESS/STRAIN SITUATION IN TUNNELS OR BORE HOLES WITH RADIOACTIVE CANISTERS EMBEDDED IN A BENTONITE/QUARTZ BUFFER MASS

DEFINITION OF PROBLEM

It has been claimed that movements in pre-Cambrian rock where high level radioactive waste products are deposited may affect the rock so that safe storage cannot be guaranteed. One reason would be that continuous openings may be created in the rock leading to an unacceptable direct connection between the ground surface and the waste products. Another reason would be that slow or sudden shear strain may destroy or break up the containers with the waste products or create large openings in the buffer mass. It is the object of this report to present the authors' main ideas concerning the possible occurrence of large unexpected movements in Swedish pre-Cambrian rock and to give the theoretical basis for the calculation of stress and strain in the canisters and the buffer mass.

MOVEMENT-PRODUCING AGENTS

The violence of certain volcanic eruptions, the mass displacements along certain faults, and the land rise in Scandinavia all illustrate the dynamics of this planet. The most unstable zones are the peripheries of the continents and certain oceanic zones. Stable areas with only very moderate internal movements are the so-called shields, one of which is the Russian Platform where Sweden is situated. The land rise and sinking (Skåne) of Sweden is the only major movement of Sweden and has been so since the end of the latest glacial period several thousand years ago. It is being associated with a low frequency of very moderate earth shocks. There are two main reasons for this movement: 1) visco-elastic recovery of the earth crust from the depressed state caused by the glacier loads and 2) large scale tectonic processes.

PRESENT AND FUTURE DEFORMATION PROCESSES IN SWEDISH PRE-CAMBRIAN ROCK

The exact location and type of presently occurring movements is not known. It may be a general slow movement of a viscous type and/or a successive step-strain on a microscopic scale, and occasionally a step-wise occurring strain on a larger scale where shear stresses have accumulated to a critical level. The last-mentioned strain behaviour generally means that very large rock units are mutually moved along a weak zone (cf. San Andreas fault) or that a large rock complex is in a condition of general failure ("tectonics").

It would be logical firstly to define the present stress situation and to determine the modes of deformation it will cause in future. A logical second step would be to assume reasonable stress changes and determine their future influence on the deformation pattern. Unfortunately there is very little information about the regional stress situation. It is believed, however, that it varies considerably from area to area and that high stress concentrations exist locally. Since the basic knowledge concerning the stress pattern is not sufficient to permit reliable calculation of future deformations another approach, which comprises the following steps, is chosen here:

- A reasonable regional stress situation is assumed

- A stress change leading to general failure of a large rock volume is superimposed
- The deformation pattern for this failure condition is determined

The regional stress situation

Case A

One extreme would be to assume an existing stress field according to HAST's measurements[1]. At the ground surface the horizontal stress can then be taken as 5-10 MPa and at 1000 m depth as 40-70 MPa. The vertical stress can be assumed to increase from zero at the ground surface to about 30 MPa at 1000 m depth.

Case B

The opposite extreme would be to assume a stress situation according to the theory of earth pressure at rest, that is the K_o condition. If we apply the theory of elasticity and put ν=0.1-0.25 we obtain a horizontal stress equal to zero at the ground surface and equal to 3-10 MPa at 1000 m depth. These values are lower than indicated by any recorded measurements. The vertical stress would still be zero at the ground surface and about 30 MPa at 1000 m depth.

The condition of general failure

To obtain a general shear failure the deviator stress must be increased to a critical level and this can be achieved either by increasing or decreasing the horizontal stress or the vertical stress. Since the vertical stress is almost entirely dependent on gravity it can not be very much changed even if long geological periods are taken into consideration. The

[1] HAST's values represent very high stresses. It is not probable that such high stresses are representative for large regions.

horizontal stress, on the other hand, can be subjected to considerable changes. It may decrease due to relaxation or to stress release in course of the land uplift and it may increase due to tectonic force fields.

A general idea about the possibility of producing general shear failure by decreasing the horizontal pressure is obtained by considering the following problem: Which is the maximum height of a free vertical rock slope in a homogeneous rock mass? Applying reasonable values of the cohesion and angle of internal friction we find the critical height to be of the order of 200-500 m. This shows that no general failure would be produced if the horizontal stress is reduced to zero down to these depths. Even if such a stress reduction will take place to much larger depths no failure can be produced unless vertical joints are opened laterally (horizontal expansion). A stress reduction of this magnitude cannot be produced unless our part of the Russian Platform is turned into an active geological area. This is not likely to occur even when we reach the state when a new geosyncline is developed at the Norwegian west-coast.

This leaves only one alternative stress situation for producing a condition of general failure: the case where the horizontal stress is increased. Since the condition of a critical deviator stress is most plausible if we assume that the presently existing horizontal stress is very high Case A should be applied.

Assuming the rock mass to be homogeneous the failure pattern will be that of Fig. 1. Applying MOHR/COULOMB's theory and the theory of plasticity we obtain:

$$\sigma_h = \sigma_v \tan^2(45+\frac{\phi'}{2}) + 2\ c'\tan(45+\frac{\phi'}{2}) \quad (1)$$

where

σ_h = horizontal stress

σ_v = vertical "

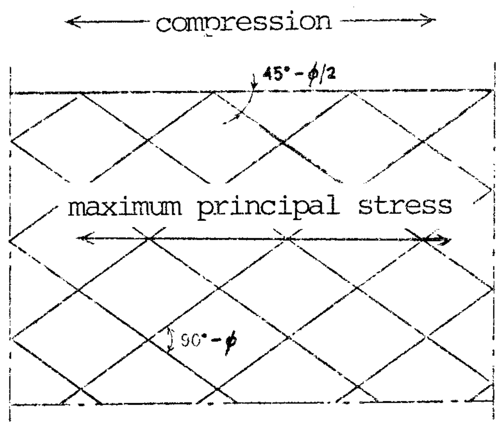


Fig. 1. Failure pattern at horizontal compression of a plastic medium

For the present purpose the MOHR/COULOMB failure criterion can be accepted for rock. It means that

$$\tau_f = c' + \sigma' \tan \phi' \quad (2)$$

where

τ_f = shear strength

c' = cohesion

σ' = effective stress (total stress minus pore water pressure) against the failure plane

ϕ' = angle of internal friction

It is obvious from Eq. (2) that the shear strength is not a characteristic constant value for any rock type. Instead, a relevant value of the shear strength can only be given if the effective pressure against the (potential) failure plane is known and applied. This requires that the stress distribution in the rock mass is known. It should be noticed that the failure envelope is in fact not a straight line as implied by Eq. (1). This is shown by Fig. 2 which gives $c' = 60$ MPa and a ϕ'-value which is about 65° in the interval $\sigma' = 0$ to 100 MPa and which decreases to about 40° when σ' approaches the very high value 600 MPa.

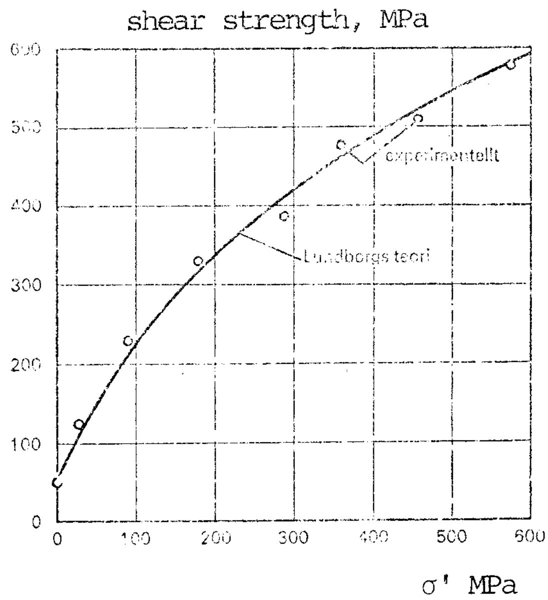


Fig. 2. Failure envelope for granite (after LUNDBORG)

The cohesion is strongly dependent on the sheared rock volume. For very small rock specimens (cores) with only Griffith cracks of the dislocation type or incomplete crystal contacts, c' is of the order of 50-70 MPa for granite and gneiss. When the sheared area increases to about 0.1 m^2 the cohesion decreases to about 10% of this value due to the increased number and size of local joints and fissures. For large rock volumes the cohesion may drop to an even smaller value. It has been shown that this value may be of the order of 0.01 to 10 MPa depending on the size of the sheared area, the orientation of the shear plane, and the frequency, art and degree of continuity of joints and fissures. For large volumes of granite poor in joints c' can roughly be taken as 1 MPa and ϕ' as 60°.

Experience shows that shearing along plane joints gives $c' = 0$ and $\phi' = 40$ to 50°. When clay zones are sheared $c' \sim 0$ while ϕ' is of the order of $10\text{-}20^{\circ}$.

Assuming first c' to be 1 MPa and $\phi' = 60^{\circ}$, which thus are reasonable values for fairly good rock, we obtain $\sigma_h = 7.5$ MPa for a rock element at the ground surface which is to be compared with the present horizontal stress 5-10 MPa according to Case A. For a rock element at 1000 m depth Eq. (1) gives $\sigma_h = 425$ MPa which is to be compared with the present horizontal stress 40-70 MPa according to Case A. We see from these approximate calculations that, if Case A is applicable, the shallow parts of any rock mass in Sweden are very highly stressed today already while the existing horizontal pressure is only about 10 to 15% of the maximum horizontal pressure at failure at 1000 m depth.

Considering the strength variation in rock masses we can repeat the calculation and apply strength parameter values which represent very bad, fissured rock: $c' = 0$, $\phi' = 45^{\circ}$. For a rock element at 1000 m depth Eq. (1) then gives $\sigma_h \sim 175$ MPa which means that even in such bad rock there is a considerable safety factor at larger depths.

On the other hand, if we take the weakest existing parts of a rock mass into consideration, that is clayey or chloritic zones for which we can put $c' = 0$ and $\phi' \sim 15^{\circ}$ as an average, Eq. (1) yields $\sigma_h \sim 50$ MPa for a rock element at 1000 m depth. This means that a failure condition exists already today in such zones which are continuous and oriented in the directions of maximum shear stresses. [The theory presupposes a critical angle of about $35\text{-}40^{\circ}$ with the horizontal plane but in the general case we must consider a pre-existence of arbitrarily oriented weak zones (a "block" complex with very steep weak zones is often assumed today)]. It also means, and this is very important, that sudden and large shear strain, which thus may take place any time at the depth where the deposition tunnels are going to be situated, will only occur along already existing continuous weak zones in the bedrock. Thus, in my opinion, the problem of finding safe regions for deposition of radioactive waste products can be solved. The requirements are:

- The deposition should take place in a rock mass with a low frequency of weak, especially clayey or chloritic zones.
- No canisters must be placed in or close to weak rock zones crossed by the tunnels.
- It is conclusive that in situ rock investigations to find and locate weak zones are essential.

It should be added that in the author's opinion HAST's values for the existing horizontal stresses (Case A) are very much higher than can be expected in rock masses which contain continuous weak zones of clayey or chloritic substances since relaxation and creep will lead to a considerable stress reduction in such materials. This means that the actual possibility of the occurrence of sudden large strain in any large rock mass in Sweden is almost non-existent.

"THE INCREDIBLE CASE"

According to the previous text, future movements will take place along pre-existing weak zones only. Yet, "the incredible case" of an unexpected shear failure through intact, high quality rock has been considered by the author as well. The investigated case is that of a circular tunnel or bore hole with a diameter d and with a centrally located stiff cylindrical canister (length 2a = 1.4 x d and base diameter b = 0.16 d) embedded by a buffer mass. The dimensions correspond, for instance, to the suggested case of a tunnel with d = 5 m, 2a = 7 m and b = 0.8 m. Shear is assumed to take place along a plane perpendicular to the tunnel through the center of the canister (Fig. 3).

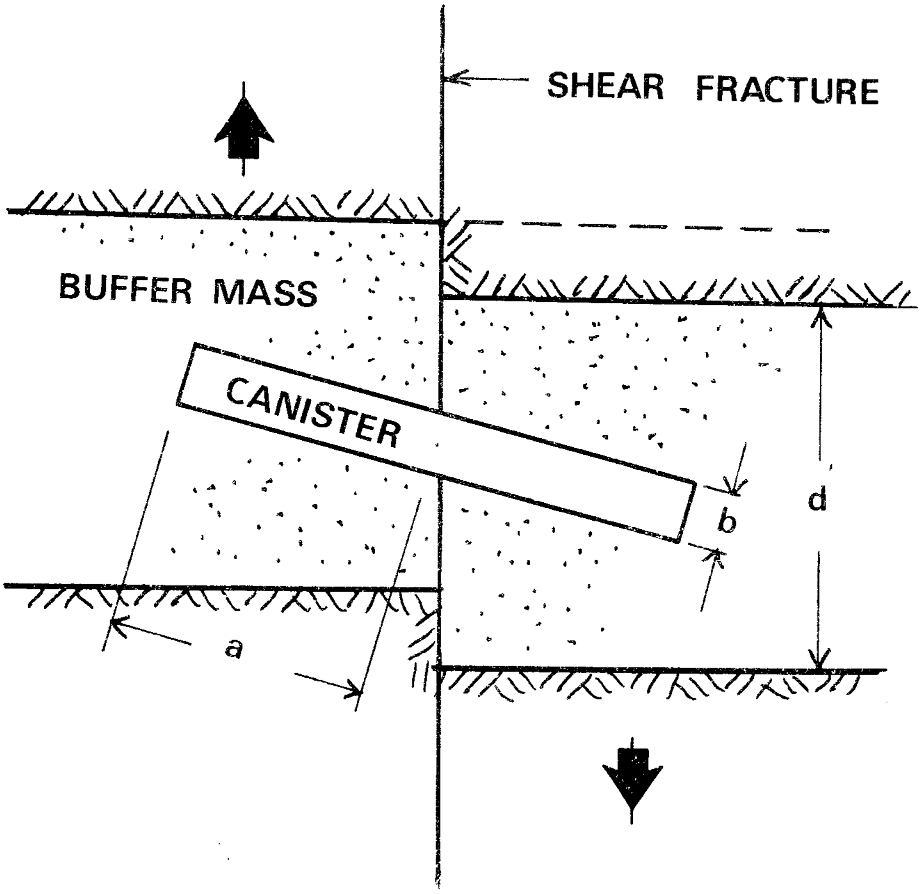


Fig. 3. The considered shear case.

What is the meaning and effect of the shear strain?

The shear strain may take place very slowly or very rapidly. In the case of slow strain the stresses in the buffer mass and the canister will be lower than if sudden shear distortion takes place. The latter case is therefore considered here. As concerns the magnitude of the strain it must be realized that a shear strain larger than a few centimeters corresponds to the case of general failure in which about the same strain is produced both down at the tunnel level and at the ground surface. An instant distortion of such a large rock mass produces vibrations which may be disastrous at the ground surface. Thus, a sudden shear strain of 20-40 centimeters will probably wipe out life and property in very large areas.
In the author's study a rapid series of sudden shear distortions of 4% of d up to a total shear strain of 50% of d has been considered. For d = 5 m this corresponds to a successive, step-wise shear distortion of 20, 40, 60, 80, 100 and 250 cm within a few hours. The consequence of such a shear process would definitely be disastrous within very large areas on the ground surface.

Test program

Two main points were considered:

- No theoretical or experimental treatment of the case shown in Fig. 3 has been presented in literature. Thus, the deformation pattern had to be estimated by some pilot test.

- Soil mechanical experience shows that scale effects concerning the stress distribution and deformation pattern are fairly small, which means that a model test would do.

The pilot/model test was made by using a simple shear apparatus consisting of two plexi-glass tubes with an internal diameter of 50 mm.

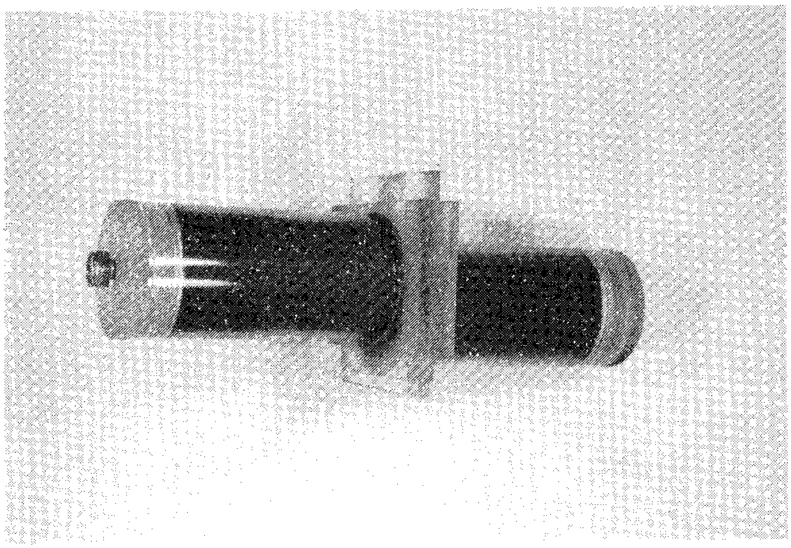
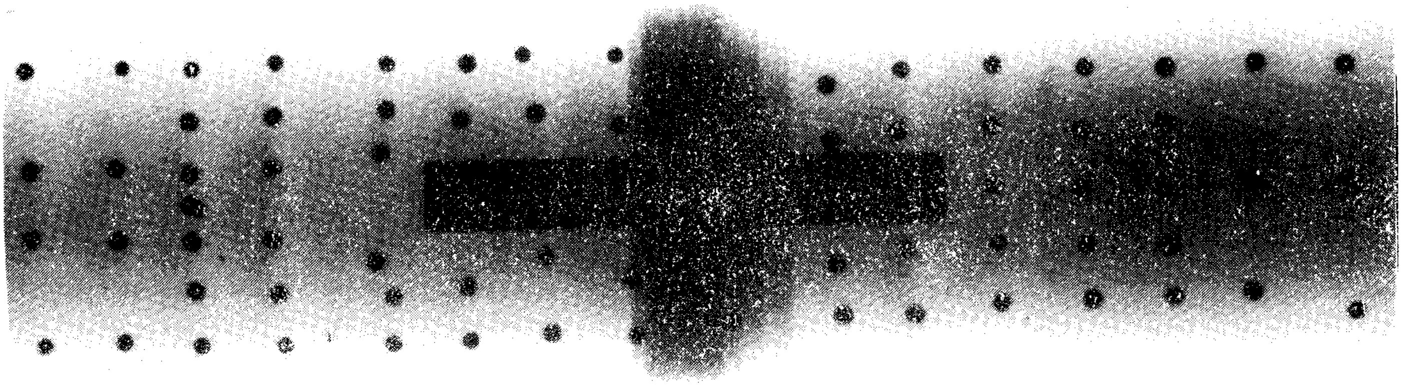


Fig. 4. The "shear apparatus". The tubes are shown at a shear distortion of 10 mm. The transparence of the plexi-glass tubes offered a good possibility of observing the diffusion process when water was taken up and to check the structural state of the mass in course of shear.

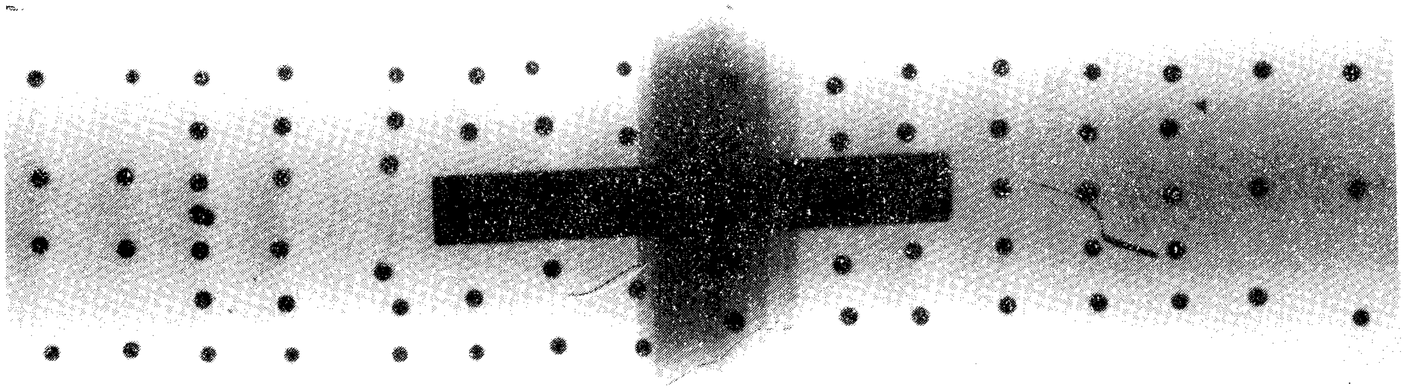
The tubes were initially in a position where their axes were coincident. They were then filled with an air-dry mass of 10% (by weight) sodium bentonite and 90% quartz[1] with a bulk density corresponding to about 1.4 t/m^3 (a fairly low density). Lead shots were also applied in the mass for X-ray determination of the deformation pattern. The canister was represented by a 0.8 cm steel axis with a length of 7 cm. The device was submerged in the standard water (cf. KBS report no. 03) until the water uptake was completed.

1) Pite silt (cf. KBS report no. 03)

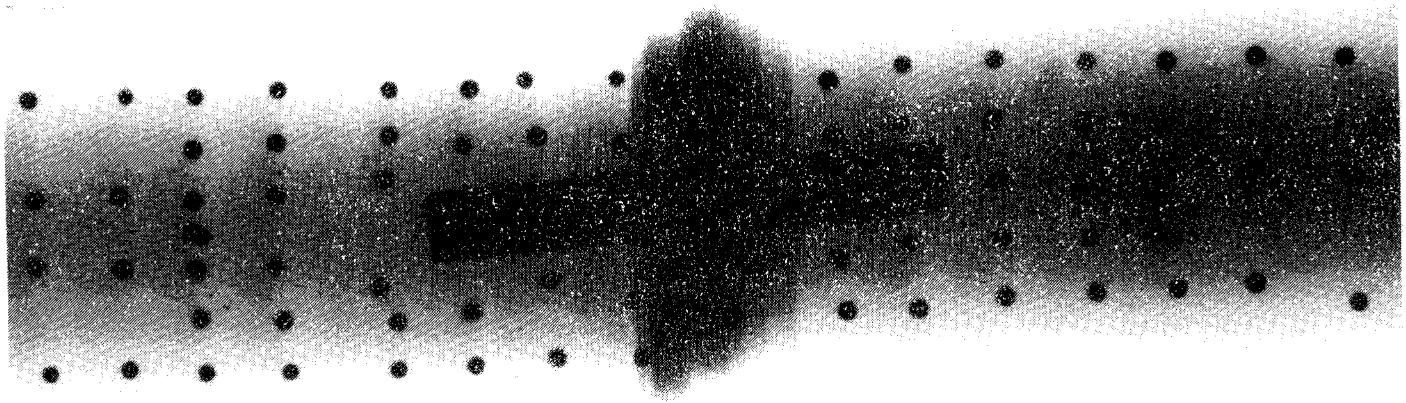
This was concluded from the observable positions of the advancing water fronts. Then, the two tubes were mutually moved to produce the rapid shear of 2, 4, 6 mm etc. and at each position an X-ray photo was taken. Fig. 5 shows a representative number of shear strain steps.



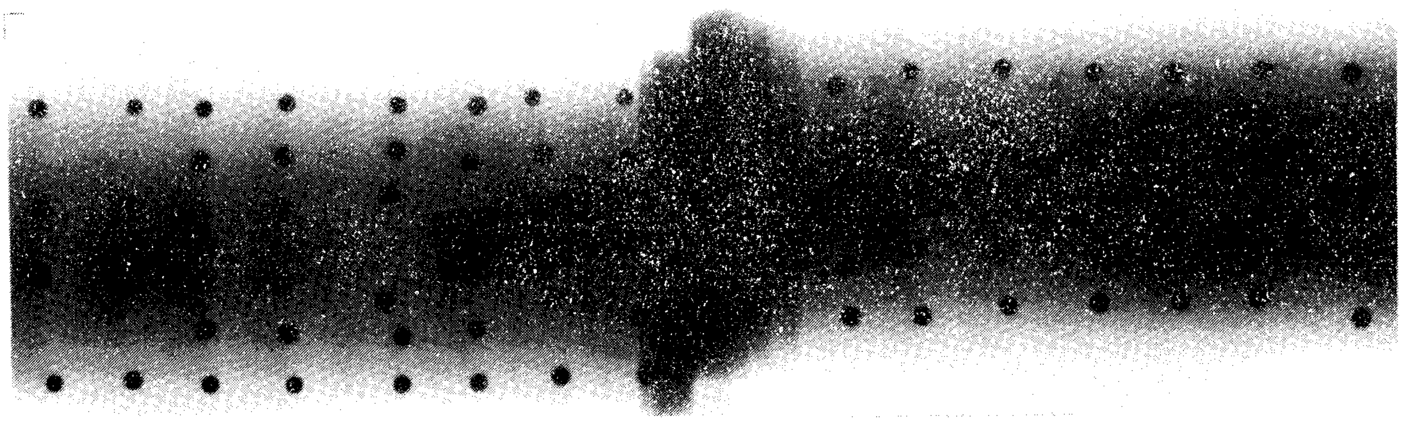
0 mm



2 mm

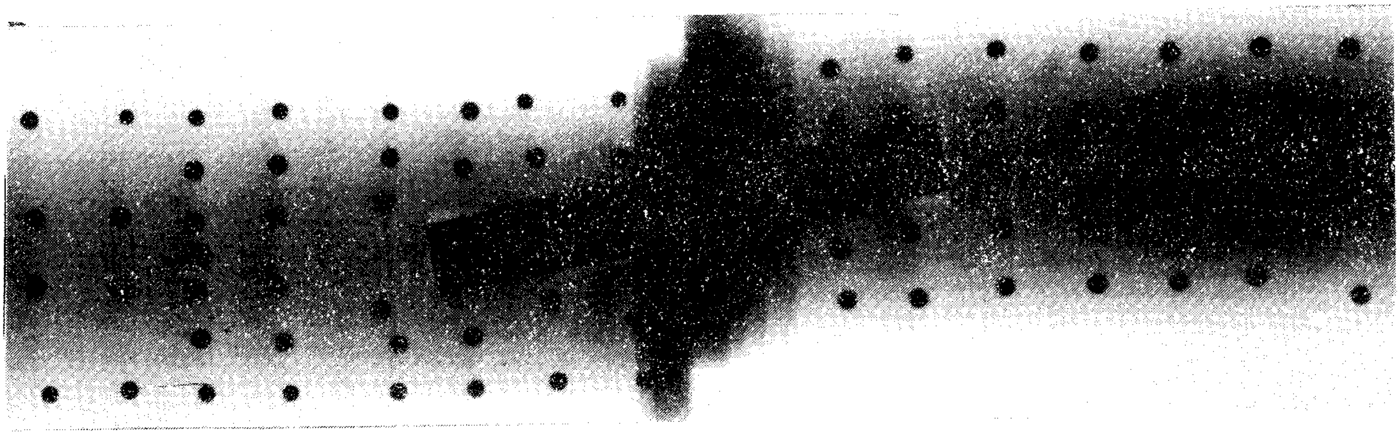


4 mm

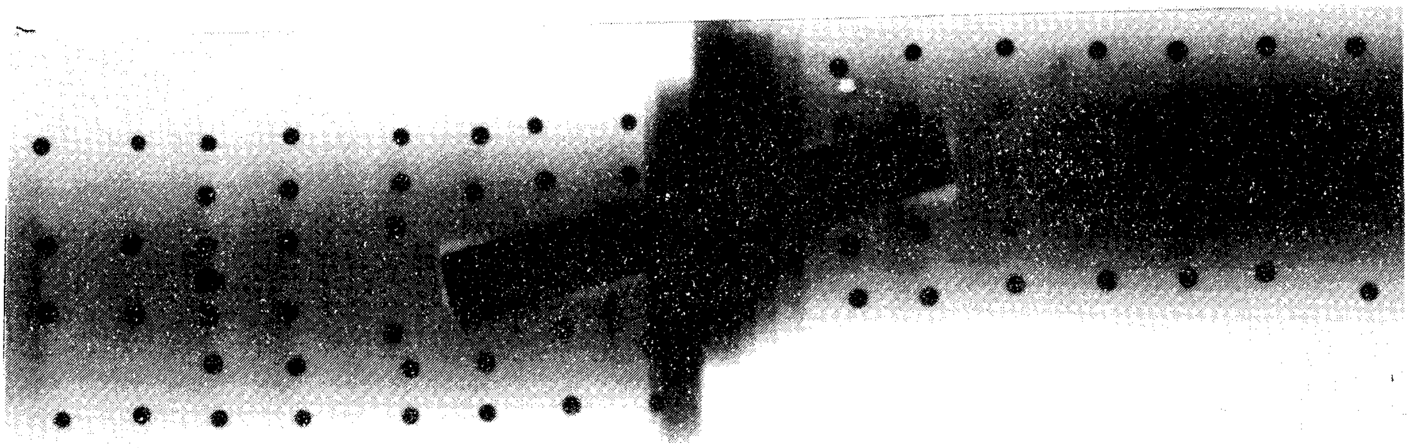


6 mm

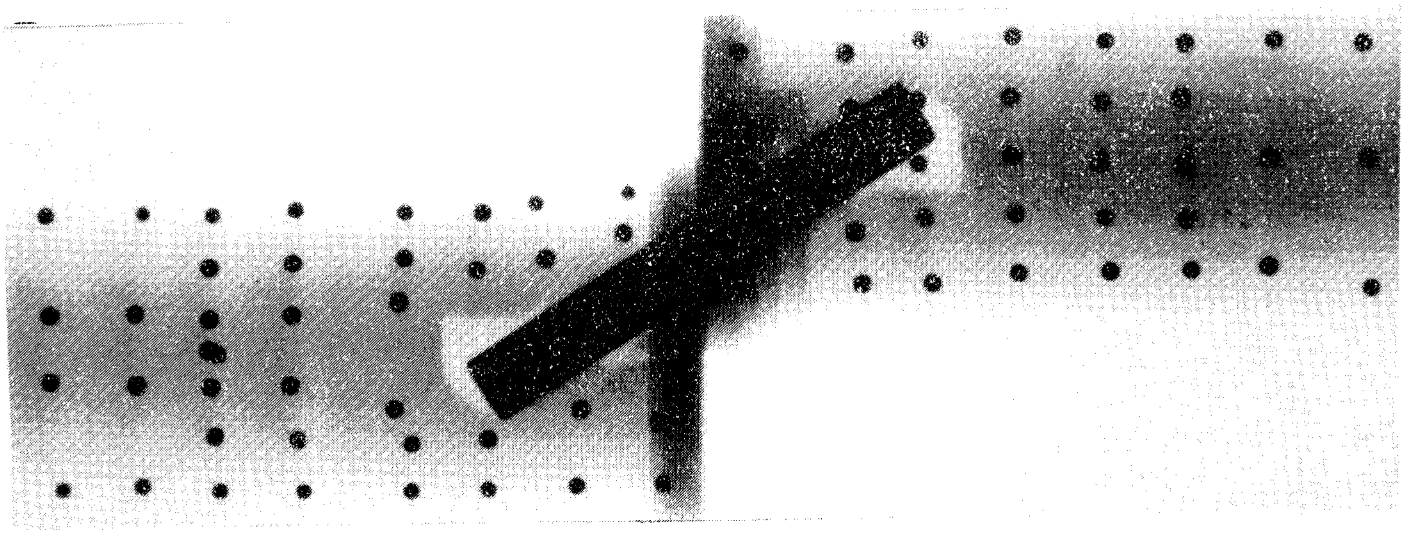
Fig. 5. See next page.



8 mm



10 mm



25 mm

Fig.5. X-ray photos at various shear strain. For d=5m
2 mm corresponds to 20 cm, 4 mm to 40 cm etc.
Notice the bright areas close to the "canisters"
at 6 mm and larger strain ("holes").

It should be noticed that at 4 mm and larger strain (corresponding to more than 40 cm when d = 5 m) the movement of the stiff canister tended to produce local empty spaces close to certain parts of the canister. These spaces are clearly seen in Fig. 5 when the shear strain has increased to 8 mm (80 cm at d = 5 m). An open space of this kind exposes the canister surface to a large pore volume but there is no connection whatsoever between these local openings and the "rock" surface. The conclusion of the study is that for d = 5 m, a sudden 40-80 cm shear distortion will not produce any appreciable change of the permeability or any other physical parameter of the system.

When the shear strain approaches 25 mm, i. e. 2.5 m at d = 5 m the shear strain is associated with a system of open shear zones in the buffer mass and there will possibly be a direct connection between the canister and the "rock" surface. Here, we must, however, again consider the effect of such an instant strain at the ground surface (large parts of southern Sweden will be destroyed!). The deformation pattern suggests that the stress distribution is of the type shown in Fig. 6 (full line). The broken line represents a reasonable approximation which is based on the assumption that the contact pressure is constant and equal to the bearing capacity q_b of a foundation slab situated deep down in a homogeneous soil mass.

The distribution and magnitude of the contact pressure shown in Fig. 6 is fairly independent of the rock shear strain which means that also moderate movement (5-10 cm for d = 5 m) produces a contact pressure of the order of the bearing capacity (ultimate contact pressure) q_b. q_b can be estimated by applying common theories concerning deep foundations but here we find largely varying results. MEYERHOF's theory

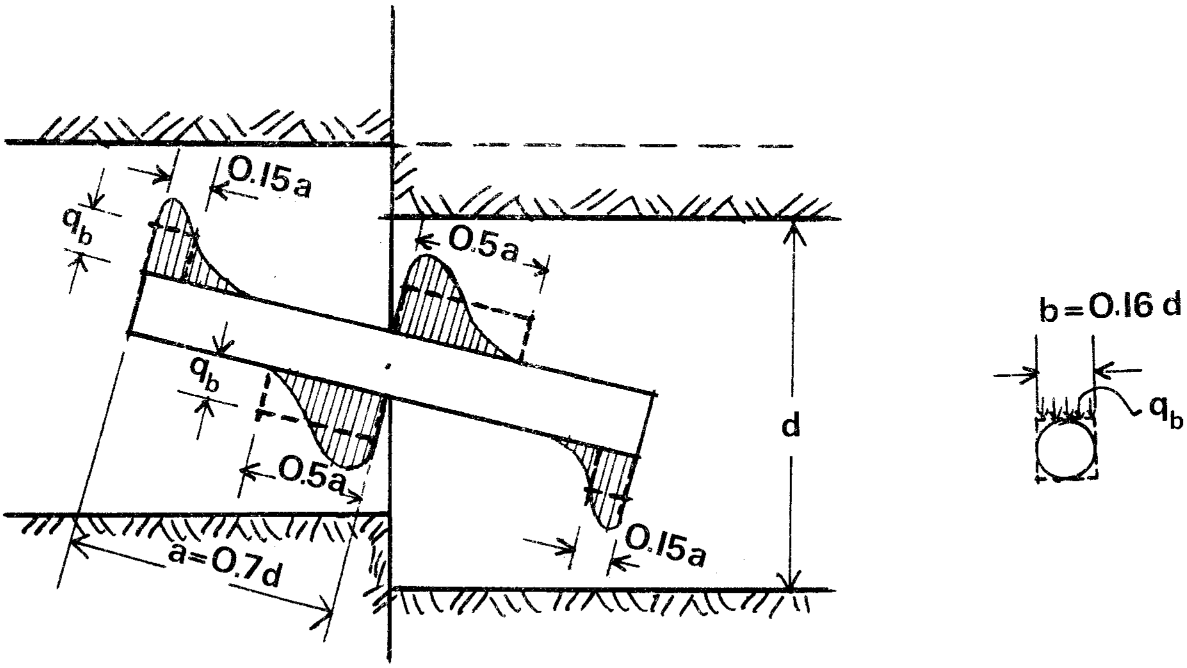


Fig. 6. Contact stress distribution.

(cf. Fig. 7) states that for frictional material such as the buffer mass (cf. KBS 1)

$$q_b = \lambda g \rho' \frac{b}{2} N_{\rho q} \tag{3}$$

where reasonable values are:

$\rho' = 1800$ kg/m³

$\phi' = 30^{\circ}$ (reduction necessary!)

$K_o = 0.4$

$\lambda = 1$

which yields $N_{\rho q} = 2500$ and $q_b \sim 3 \cdot 10^4 b$ kPa.

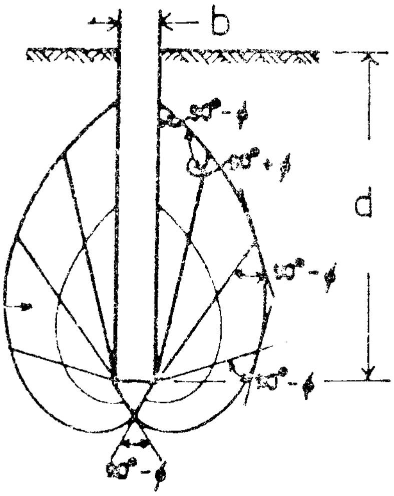


Fig. 7. Failure pattern according to MEYERHOF.

BEREZANTZEV's theory (cf. Fig. 8) states that

$$q_b = g\rho' \frac{b}{2} N_{kc} \tag{4}$$

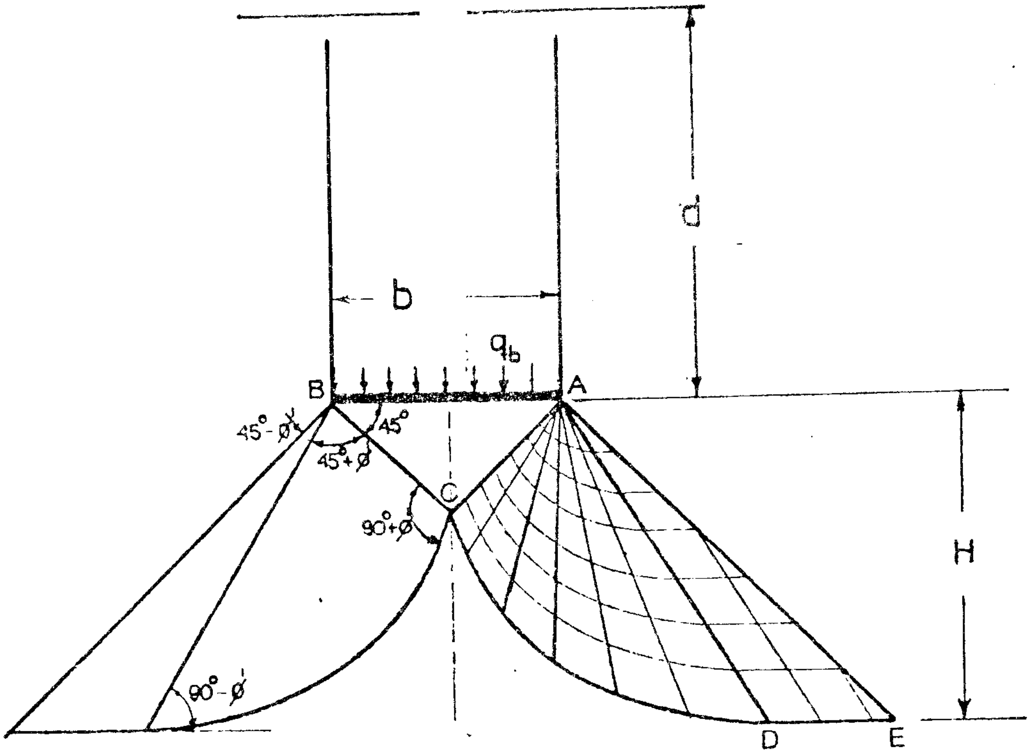
where reasonable values are:

$\rho' = 1800 \text{ kg/m}^3$

$\phi' = 35^{\circ}$

$\frac{d}{b} = 4$

which yields $N_{kc} = 250$ and $q_b \sim 3 \cdot 10^3 b$ kPa.



$$H = \frac{B}{\sqrt{2}} \cos\phi' \, e^{(\frac{\pi}{4} + \phi')\tan\phi'}.$$

Fig. 8. Failure pattern according to BEREZANTZEV.

Since no experimentally determined q_b-values are at hand it is necessary at present to use the least favourable value, that is MEYERHOF's $q_b = 3 \cdot 10^4 b$ kPa (where b has the dimension meter) although this expression yields very high, almost improbable values.

The calculation of the maximum bending momentum (about $0.075 \cdot q_b \cdot a^2 b$ at a distance of about 0.3 a from the canister's center) and the maximum shear force (about $0.35\ aq_b \cdot b$ at the canister's center) is readily made by applying Fig. 6.

It is interesting to see that for d = 5 m, b = 0.8 m and a = 3.5 m, the maximum bending momentum will be about 22000 kNm which, for a circular homogeneous canister section, produces a maximum tensile stress of about 440 MPa and a maximum shear stress of about

47 MPa. This illustrates that very high stresses are produced in stiff canisters under the geometrical conditions considered here. High-quality steel can sustain such stresses while copper can not. It is quite obvious, however, that the stresses can be largely reduced if the geometry is changed for instance so that a[1] will be equal to 0.3 to 0.5 d instead of 0.7 d. Since such changes will again affect the stress distribution (probably not very much, however) a thorough, systematic investigation is required to obtain general relationships. Such an investigation should also comprise the asymmetric case, that is where the shear plane does not pass through the center of the canister.

Roland Pusch Luleå 1977-08-22

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1) Definition according to Fig. 3.

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