
KBS TEKNISK RAPPORT

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buffer substance**

Roland Pusch

Högskolan i Luleå 1978-02-25

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TEKNISK RAPPORT **KBS 13**

REPORT ON

HIGHLY COMPACTED

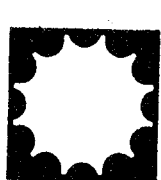
Na BENTONITE AS BUFFER

SUBSTANCE

Luleå 1978-02-25

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MASKINTEKNIK • GEOTEKNOLOGI • ARBETSVETENSKAP

HIGHLY COMPACTED Na BENTONITE AS BUFFER SUBSTANCE

SCOPE OF INVESTIGATION

The possible use of highly compacted sodium bentonite was first suggested by HANNERZ (ASEA Atom) and a number of test pieces (bentonite MX-80) have now been investigated by several laboratories in various respects, especially concerning the water uptake, the swelling pressure, and the diffusion properties. The use of highly compacted bentonite as a buffer substance in bore holes requires an investigation of the combined effects of and on the material in situ. A number of data are collected and presented here for this purpose. Explicitely, this report deals with:

- Definition of highly compacted bentonite
- Experimentally and theoretically determined mechanical and physical properties of highly compacted bentonite
- Technical aspects of the in situ behaviour of highly compacted Na bentonite buffer mass

DEFINITION OF HIGHLY COMPACTED BENTONITE

ASEA Atom's test pieces as well as those produced at the Division of Soil Mechanics, University of Luleå, are made by compacting an air-dry powder of a standard quality bentonite MX-80 Volclay (Wyoming bentonite) from the American Colloid Company. Pressures in the range of 50-100 MPa have been used for the compaction in a number of tests and densities from about 2 to 2.3 t/m³ have been reported. A specific pressure is not a criterion for obtaining identical samples since the bulk density depends on

the porosity as well as of the water content. Thus, a very dry bentonite powder requires a higher pressure to obtain a certain bulk density than a moist powder. The water content of the powder must therefore be specified with great care.

The required granular, mineralogical and chemical properties of the bentonite powder will not be discussed in this report. It is suggested, however, that MX-80 powder is taken as a reference substance since any bentonite intended for the production of highly compacted buffer substance bodies should have properties equal or similar to those exhibited by the MX-80¹⁾. As mentioned previously the water content of the bentonite powder must also be defined. It is suggested here that the water content is taken as 10%.

Then, by specifying a certain compacting pressure, the bentonite bodies are sufficiently well defined with reference to porosity and microstructural properties and their behaviour as concerns permeability, swelling pressure etc can be safely predicted. It is suggested here that a compacting pressure of 50 MPa is used.

MECHANICAL PROPERTIES OF HIGHLY COMPACTED Na BENTONITE

The mechanical properties are of central interest for the stage of handling, transportation and deposition as well as for the subsequent very long period of time when the bentonite will serve as a buffer mass.

¹⁾ This does not exclude the use of a number of natural bentonites or artificially produced montmorillonite-like clays.

The present study concerns the following properties:

- Density
- Swelling pressure as a function of density
- Permeability as a function of density
- Stress/strain relationship as a function of density
- Strength as a function of density

Density

The specific density is a function of the mineral composition. The MX-80 material is largely dominated by montmorillonite which, hence, determines the density. For montmorillonite literature provides values between 2.4 and 2.8 t/m³, the scattering being due to a varying definition of the amount of water firmly held to the clay lattice. The author has used 2.7 t/m³ for his calculations.

The bulk density is defined as the ratio of total weight and total volume. It depends on the water content, w%, which is expressed as the ratio of water weight and weight of solid (mineral) substance. The degree of water saturation ($S_r\%$) tells us to what extent the voids are filled by water. It is expressed as $\frac{V_w}{V_p}$ where V_w is the water volume and V_p is the pore volume. The porosity is often expressed in terms of the void ratio $e = \frac{V_p}{V_s}$ where V_s is the mineral volume. The void ratio, bulk density and water content of sodium montmorillonite (and approximately of sodium bentonite) are related according to Table 1 for completely water saturated material.

Table 1. Bulk density and water content (100% water saturation) versus void ratio

Void ratio e	Bulk density ρ_t/m^3 (g/cm ³)	Water content w%
0.1	2.54	3.7 ¹⁾
0.2	2.42	7.4 ¹⁾
0.3	2.31	11.1 ¹⁾
0.4	2.21	14.8
0.5	2.13	18.5
0.6	2.06	22.2
0.7	2.00	25.9
0.8	1.95	29.6
0.9	1.90	33.3
1.0	1.85	37.0
1.5	1.68	55.5
2.0	1.57	74.1
3.0	1.43	111.1
4.0	1.34	148.1
5.0	1.28	185
10.0	1.15	370
20.0	1.08	741

¹⁾ These values do not correspond to a complete monolayer. Water saturation is probably achieved only for water contents larger than about 25%.

Swelling pressure

If MX-80 powder with a water content of 10% is compacted under a pressure of 50 MPa experience shows that the bulk density will be about 2.05 t/m³ while about 90 MPa is required to increase the density to about 2.2 t/m³.

For the first-mentioned bentonite the degree of water saturation equals 60% and it will be water saturated when the water content is increased to about 16%. The bulk density is thereby increased

to almost 2.2 t/m^3 . For the bentonite compacted at 90 MPa the degree of water saturation equals 78% and it will be water saturated when the water content is increased to about 13%. The bulk density is thereby increased to 2.27 t/m^3 .

It is well known that if a compacted bentonite body is confined so that it cannot swell it will exert a pressure - swelling pressure - if exposed to external water (through a filter). The physical explanation of this phenomenon is complex. Part of the swelling pressure is due to osmotic forces which create a suction but other phenomena are also involved. It has been dealt with for decades in colloid chemistry and various applied soil sciences and is discussed in some detail by PUSCH (1978) who recommends a simple method for calculating the swelling pressure of water saturated Na montmorillonite. The application of the method yields the extrema (curves B and D) in Fig. 1. Curve B represents the special case of 100% Na montmorillonite with all particles oriented in a parallel fashion so that maximum swelling pressure is produced in one direction (anisotropic state). This microstructural order can only be obtained when a clay slurry is compacted from the wet side. Curve D represents the case of 100% Na montmorillonite where 1/3 of all the particles are parallel (tactoid-type material, isotropic state). The D-curve should give a conservative estimate of the swelling pressure for the MX-80 bentonite compacted in an air-dry ($w \sim 10\%$ condition).

The observed swelling pressure of MX-80 bentonite bodies is higher, however (curve C¹), HYDÉN), and this is due to the elastic strain of particles and particle aggregates produced by the compaction. Thus, part of the pressure required to keep the volume of the compacted body constant is used to

1) The curve shows the author's recalculated relationship for complete water saturation.

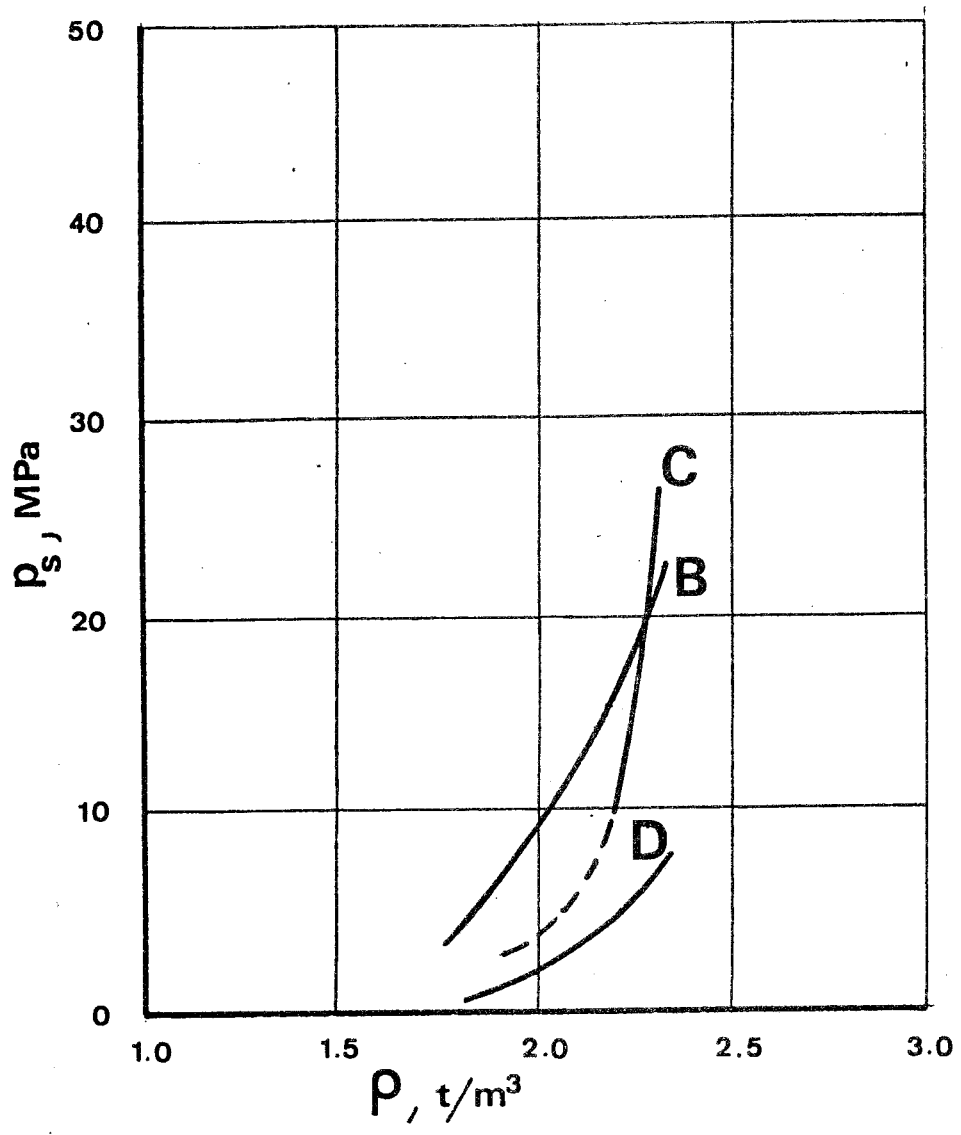


Fig. 1. Swelling pressure (P_s) versus bulk density (ρ) for water saturated Na montmorillonite (PUSCH, 1978).

compensate for the elastic energy stored in the compacted substance. This energy will dissipate slowly through a relaxation (creep) process and it disappears in the course of time. Some tens or hundreds of years will probably be required for this dissipation. Yet, it is reasonable to use the C-curve as a basis for estimating the swelling pressure since it will give values on the safe side.

We can therefore state that the compaction of MX-80 powder with a 10% water content ($w = 10\%$) gives bentonite bodies with a density of 2.05 t/m^3 and a swelling pressure of 10 MPa when the water saturation increases to 100% (by which the bulk density increases to 2.2 t/m^3). If the water saturated bentonite is allowed to swell the swelling pressure can be assumed to be reduced according to Table 2.

Table 2. Swelling pressure versus bulk density for water saturated Na bentonite.

Bulk density	Swelling pressure MPa
2.2	10
2.0	4
1.8	2
1.6	1.5
1.4	1
1.2	0.5

Permeability

Literature provides considerable information concerning the permeability of water saturated Na bentonite of various density. The matter is discussed in some detail by PUSCH (1978) who gives the general relationship between permeability and bulk dense (water saturated condition) shown in Fig. 2.

It is obvious from this figure that bentonite with a bulk density of 2 t/m^3 or more is practically impervious.

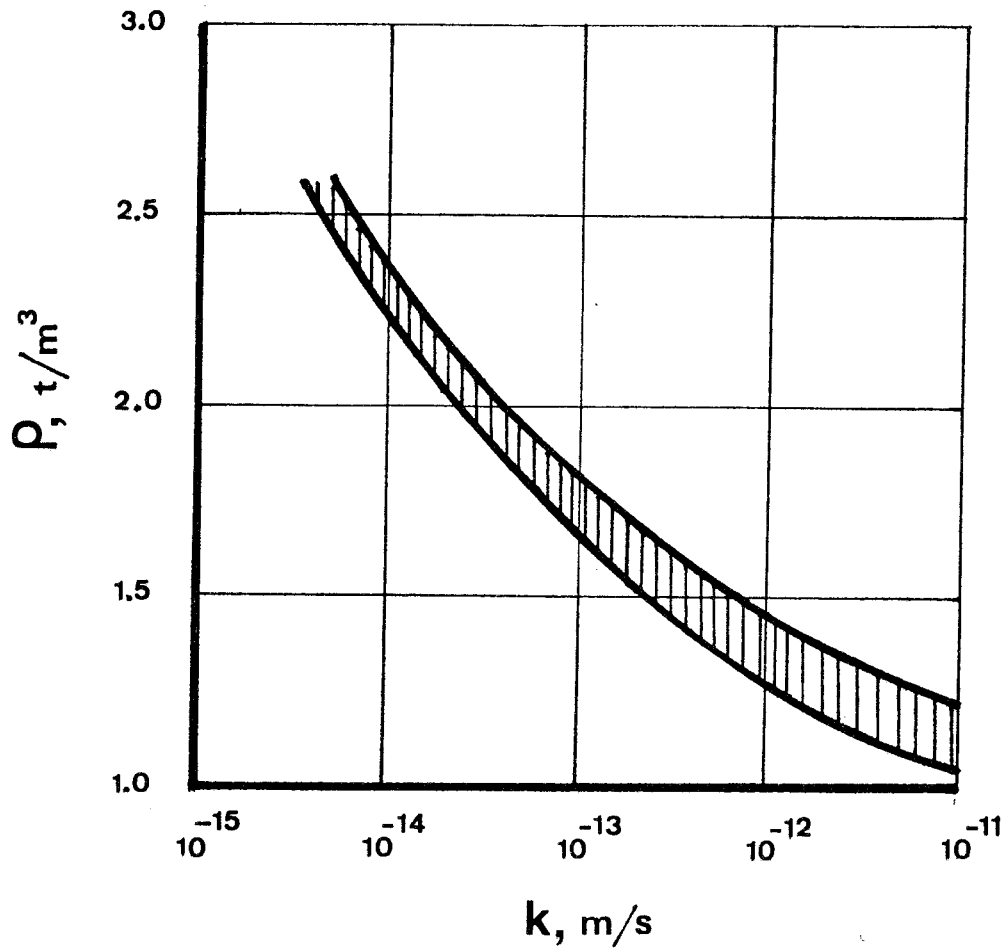


Fig. 2. General relationship between bulk density (ρ) and coefficient of permeability (k) for Na bentonite. The band shape is chosen here to take into account the influence of varying pore water salinity and scattering of experimental data.

Stress/strain relationships

The deformation properties, especially with respect to time, of highly compacted bentonite is of interest as concerns:

- Settlement of canister
- Rate of extrusion of bentonite into joints which are widened or formed in the surrounding rock
- Swelling, where expansion is possible (extrusion of bentonite into compressible tunnel fill, for instance)

The degree of water saturation and the bulk density are the major factors which affect the stress/strain relationship. For "air dry" MX-80 bentonite compacted at 50 MPa to $\rho \sim 2.05 \text{ t/m}^3$ uniaxial compression tests (Figs. 3 and 4) gave $E \sim 300 \text{ MPa}$ and $\nu \sim 0.15$. The viscosity of the bentonite in this only partly water-saturated state is hardly relevant for the estimation of canister settlement and rate of extrusion of bentonite into joints. No experimental evidence of the viscosity of highly compacted bentonite is at hand but it can be roughly estimated by considering the flow properties of other clays. The time-dependent deformation properties of a number of soft clays determined by simple uniaxial compression tests with a dead load producing a pressure corresponding to about 30% of the ultimate pressure are shown in Fig. 5. For the inorganic specimens the following relationship is approximately valid:

$$\epsilon = \frac{\sigma}{E} + \frac{\sigma}{\eta} \log t \quad (1)$$

where

ϵ = (relative) compression (dimensionless)

σ = axial stress

η = viscosity

t = time

For these soft clays with a shear strength of only 10-20 kPa η is of the order of 10^6 - 10^7 Ns/m^2 . For moraine clays it is known to be 10^2 to 10^4 times higher. Since water saturated bentonite with $\rho > 2 \text{ t/m}^3$ is even stiffer it is reasonable to believe that its viscosity is higher than 10^{10} Ns/m^2 ($>10^{11}$ pois).

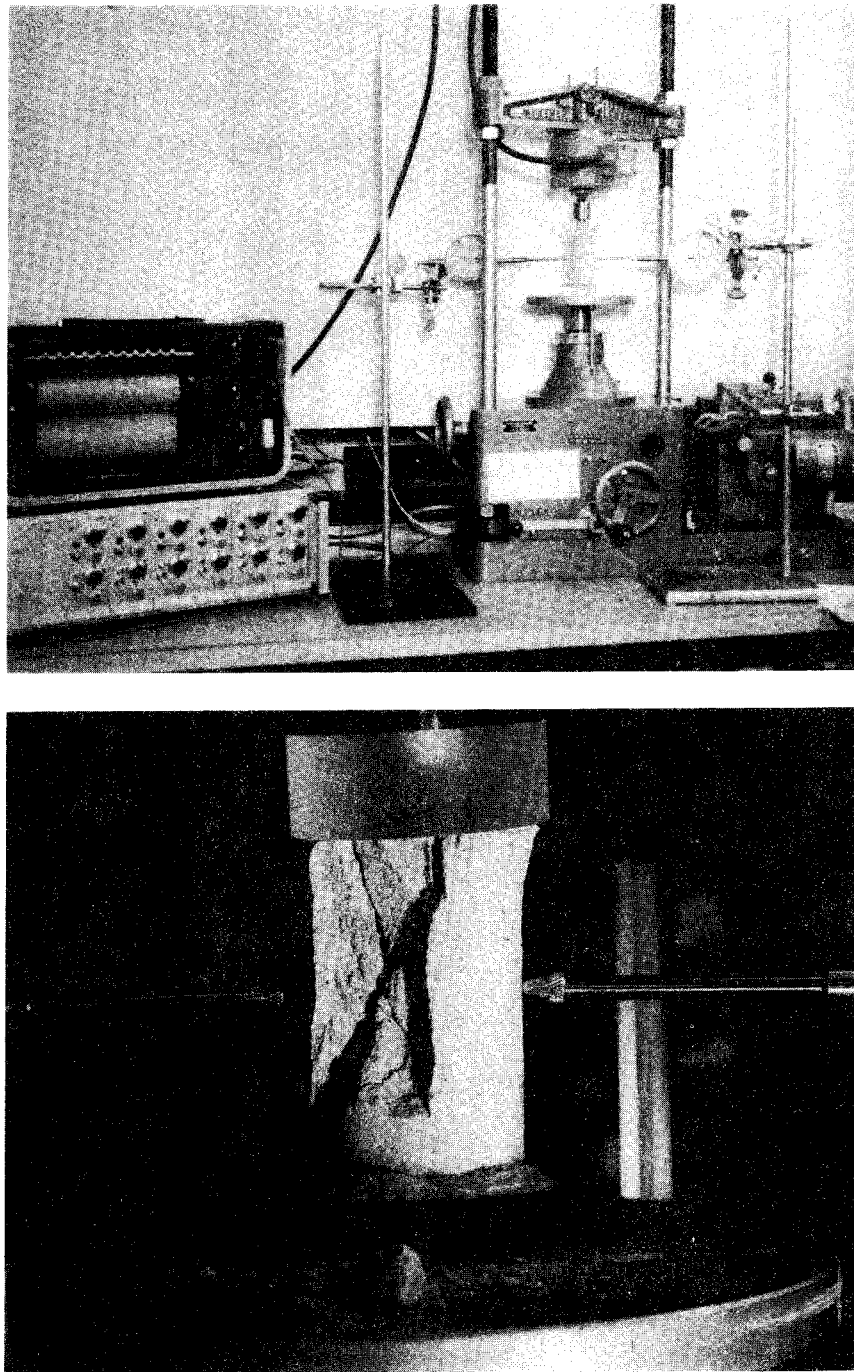


Fig. 3. Uniaxial CRS test of highly compacted bentonite. Upper picture: Loading machine and electronic recording of the stress. Lower picture: Close up of fractured specimen showing typical brittle failure.

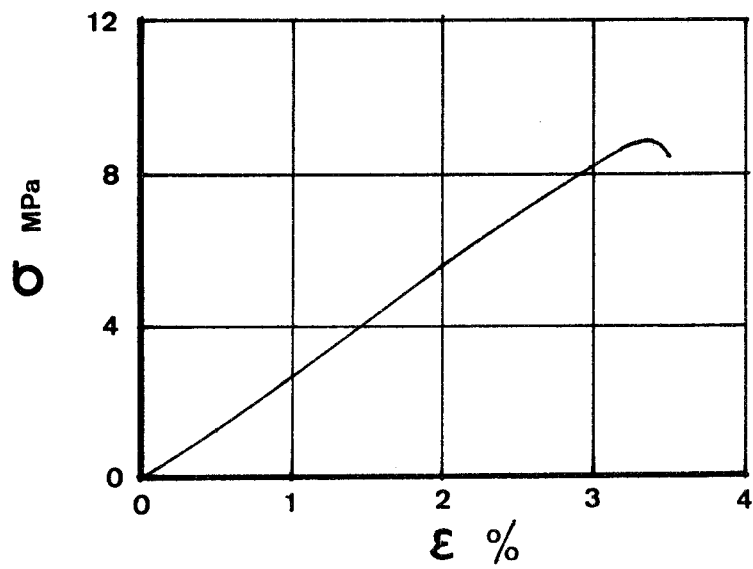


Fig. 4. Compression versus axial stress for uni-axial constant rate of strain test of cylindrical specimen with 39 mm diameter and 70 mm height.

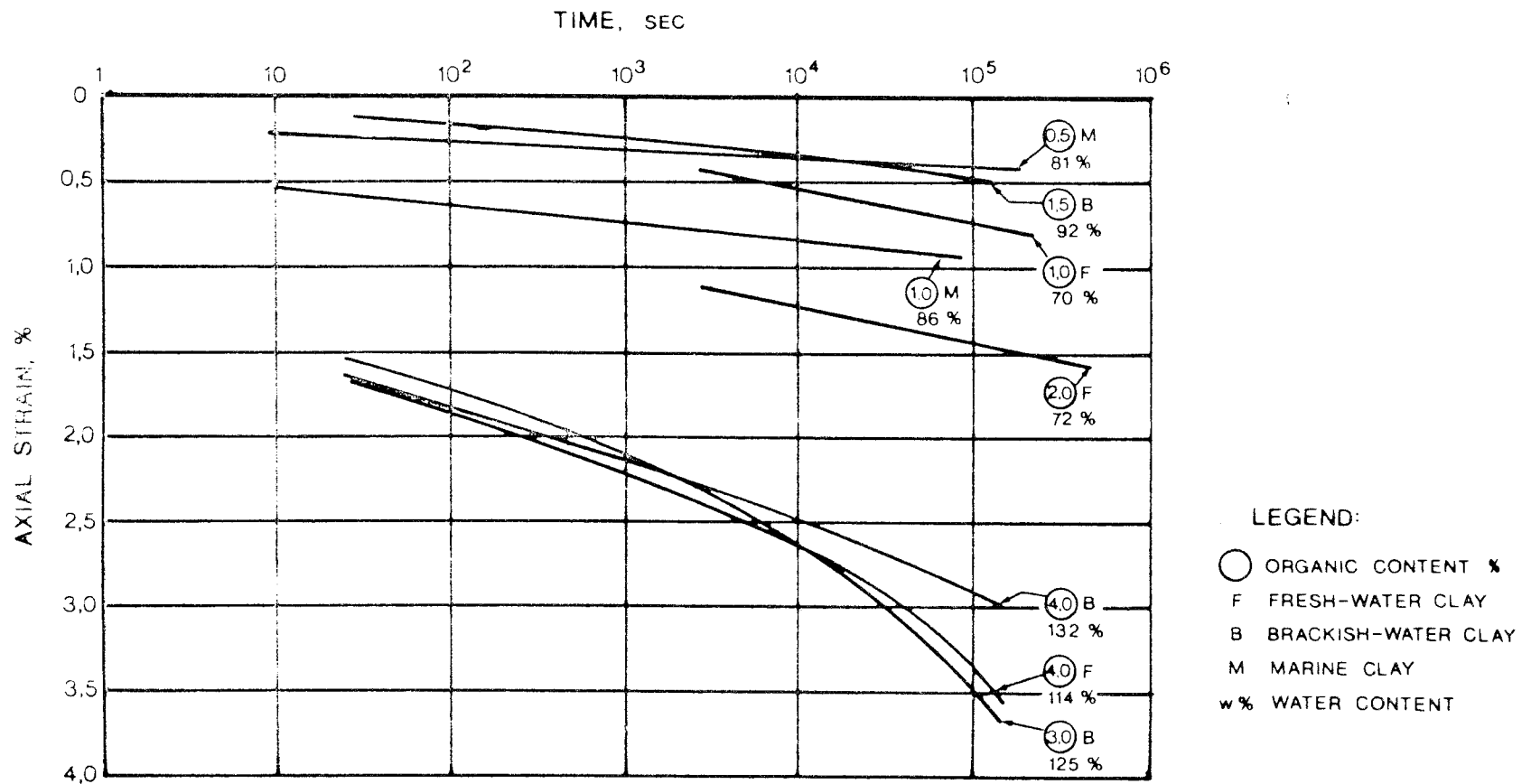


Fig. 5. Axial strain versus time at uniaxial, unconfined loading of clay specimens with a shear strength of 10-20 kPa. Upper group represents clays with low or medium organic content (PUSCH, 1973).

All these properties, the E-modulus, the ν -value, and the viscosity η , are changed when the water saturated body is allowed to swell in connection with the uptake of water. E and η will be reduced while ν increases. According to the American Colloid Company a largely increased water content ($w > 200\%$) gives the clay the character of a gelatinous paste while further addition of water to $w > 1500\%$ turns it into a milky, flowable liquid.

For water saturated bentonite with a bulk density less than about 1.5 t/m^3 , that is a condition where the viscous properties become obvious, E and ν cannot be readily defined or determined. The viscosity, on the other hand, is easily measured but very few values have been reported in literature (Fig. 6).

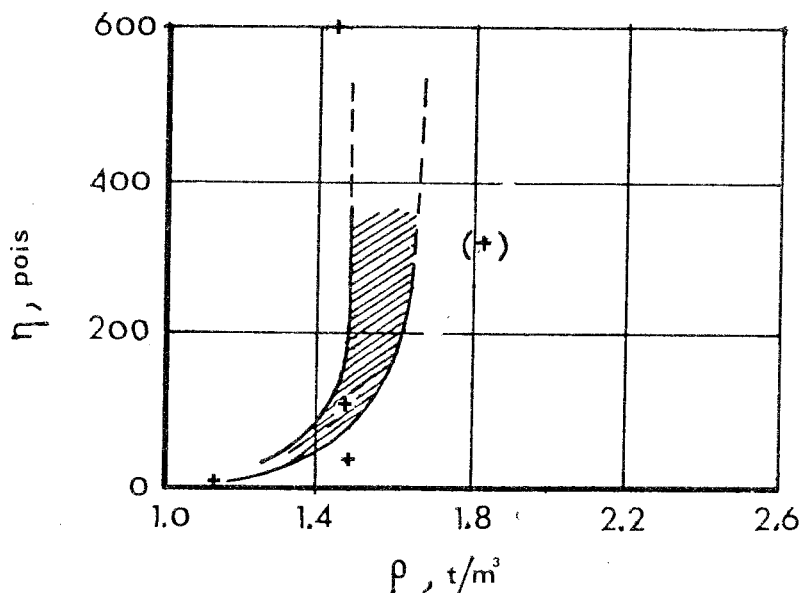


Fig. 6. Viscosity versus bulk density for Na bentonite. Hatched area represents probable zone of validity.

For further information the reader is referred to PUSCH, 1978.

For a general classification it is sufficient to identify grossly the bulk density intervals which correspond to a solid (highly viscous/elastic) and a semiliquid behaviour, respectively. This is discussed in the subsequent chapter. A more detailed definition and quantitative determination of the strain is given later in this report where the settlement of canisters and various swelling mechanisms are discussed, "self-injection" being one example.

Strength

The strength of the highly compacted bentonite is important as concerns:

- The bearing capacity of the "air dry" bentonite bodies. The relevant parameter is the shear strength.¹⁾

- The bearing capacity of the water saturated and somewhat swollen bentonite.
Again, the shear strength is the relevant parameter.

Fig. 4 tells us that the uniaxial compressive strength is 8.9 MPa for the "air dry" bentonite ($w \sim 10\%$) with a bulk density of 2 t/m^3 . The shear strength is thus about 4.5 MPa for this material.

¹⁾ The maximum tensile stress is important in connection with handling (lifting operations etc) and transportation. It is not considered here since it is part of a larger complex of problems which has to do with safe handling of the bentonite bodies during their deposition (cf. next chapter).

If the "air dry" bentonite takes up additional water leading to 100% water saturation in a perfectly confined condition its strength may be a little reduced since the capillary forces which contribute to the grain contact pressure disappear and the average particle distance increases. The reduction should be very moderate, however, since the degree of water saturation of the "air dry" substance is already of the order of 60%. If it is allowed to swell, still completely water saturated, the decreased bulk density and increased water content means that such strength-producing factors as the dilatancy and the average number of particle bonds decrease. This reduces the shear strength as is also evident from the reduction of the viscosity when the bulk density decreases. In this case literature is of no help since relevant shear strength values have hardly been reported as far as the author knows. The reason for this is probably that shear tests are very difficult to perform, especially at high bulk densities, because of the very high swelling pressures. The problem of estimating sufficiently reliable shear strength values can be solved, however, by applying basic theoretical soil mechanics.

This requires that the physical meaning of swelling pressure can be understood and correctly interpreted. In the author's opinion this pressure is equivalent to the effective ("grain contact") pressure. The general expression for the shear strength of cohesive soils is:

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

where τ_f = shear strength
 c' = cohesion
 σ' = effective pressure
 ϕ' = angle of internal friction

If $c' = 0$, which is a conservative approach, and ϕ' is taken as low as 10° we obtain since $\sigma' = p_s$ (i.e. the swelling pressure):

$$\tau_f = 0.16 p_s \quad (3)$$

where p_s is the swelling pressure. The very low value of the angle of internal friction is certainly a very safe assumption for high densities (swelling pressures) because it does not account for dilatancy. On the other hand, it is probably fairly correct for low densities.

For the water saturated bentonite with $\rho = 2.2 \text{ t/m}^3$ and $p_s = 10 \text{ MPa}$ Eq. 3 yields $\tau_f = 1.6 \text{ MPa}$ which is a shear strength typical of soft (wheathered) sedimentary rock. Table 3 gives the calculated shear strength for a number of bulk densities.

Table 3. Shear strength versus bulk density of water saturated Na bentonite.

Bulk density $\rho, \text{ t/m}^3$	Shear strength $\tau_f, \text{ kPa}$
2.2	1 600
2.0	640
1.8	320
1.6	240
1.4	160
1.2	80
1.1	$\leq 30^1)$
1.05	$\leq 15^1)$

1) The meaning of a shear strength of such soft substances can certainly be debated. The values are given here to illustrate the soft state of very dilute clay.

It is interesting to see that the previously mentioned

water content values 200 ($\rho \sim 1.25 \text{ t/m}^3$) and 1500% ($\rho \sim 1.04 \text{ t/m}^3$) which correspond to certain degrees of soft consistency are in reasonable agreement with Table 3. For comparison it can be mentioned that undisturbed, soft natural Swedish clays generally have a shear strength of 10-15 kPa ($\rho \sim 1.4-1.5 \text{ t/m}^3$), while for heavily consolidated moraine clays with $\rho > 2 \text{ t/m}^3$ we find τ_f -values higher than 100 kPa.

TECHNICAL ASPECTS OF THE IN SITU BEHAVIOUR OF
HIGHLY COMPACTED Na BENTONITE BUFFER SUBSTANCE

Water uptake and swelling of the highly compacted bentonite affects its mechanical and physical properties and the stress situation in the confining rock. A number of questions which concern these processes and their important consequences are considered in this chapter. Fig. 7 forms the basis for the discussion.

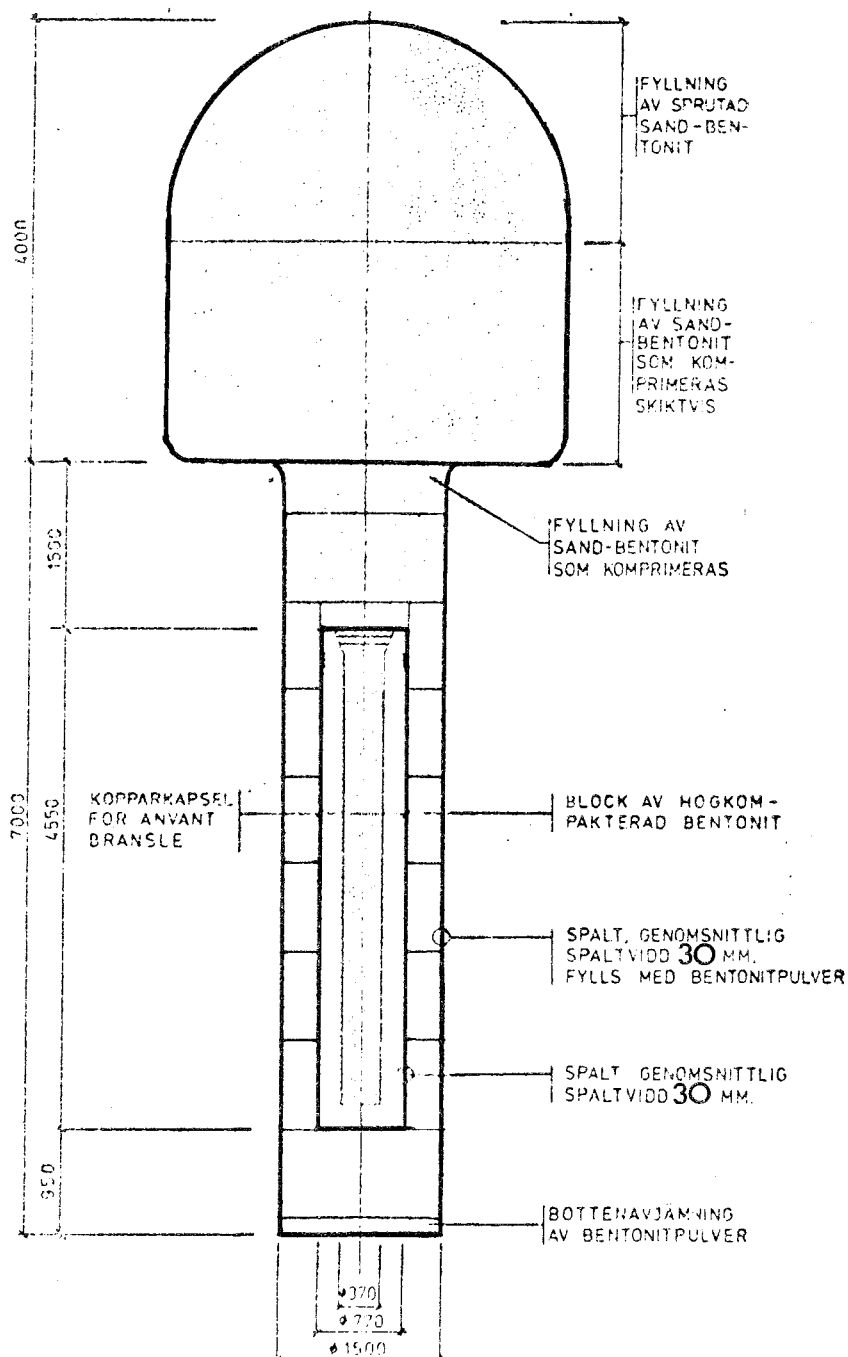


Fig. 7. Schematic picture of tunnel and deposition hole (VBB).

The following subjects are considered:

- Handling of compacted bodies.
- The water uptake in the bentonite; rate, uniformity.
- Development of swelling pressure in bore holes and tunnels.
- Geotechnical properties of the water saturated bentonite (bearing capacity, settlement of canister).
- Stress/strain effects.

Handling of compacted bodies

It is obvious that highly compacted bentonite bodies do not disintegrate in ordinary atmosphere with a (relative) humidity of 40-60% even after several months. Cracks appear, however, after a few weeks already and they affect the tensile and compressive strengths and therefore also the possibility of handling the bodies. The cracking is probably due to stress concentrations caused by the compaction. A slightly non-uniform water uptake may also be responsible for local fissuring. It must of course be guaranteed that the bentonite bodies do not fall apart or loose fragments during the deposition in the bore holes. Otherwise the bodies may be stuck with concomittant serious trouble. In the author's opinion this can be safely solved by placing the plant for the production of compacted bodies down in the deposition area (short time passed between production and deposition), by using reinforcement bars (grids) of copper in the compacted bentonite bodies, and by protecting the bodies temporarily with a metal casing during their deposition.

Water uptake in the bentonite

Physics

The physical processes involved in the water uptake are rather complex. The subject is dealt with in some detail by PUSCH (1978) in a report on bentonite extrusion into rock joints. The "air dry" highly compacted bentonite bodies contain pores which are filled by water to about 60%. In this state the cohesive effect of the tremendous capillary forces maintains the high density and keeps the volume constant. If such a dense body is submerged in water, water molecules start to move, rapidly at first, into the body, the transport probably taking place in continuous water films along the mineral surfaces in the partly saturated clay. This process takes place with minor volume increase only and thus without the formation of noticeable swelling pressure. When the degree of saturation becomes high and capillary forces are less active, water is transported and taken up mainly by osmosis and other processes ("water crystallization"). The driving force for this water transport is the suction which is an equivalent of the swelling pressure (with the opposite sign). When the clay is water saturated large amounts of additional water can be taken up if the clay is free to swell. Water saturation of a confined bentonite body produces a swelling pressure as discussed previously. The suction is successively reduced in a swelling bentonite body but a condition of equilibrium is not obtained until large amounts of water have been absorbed which leads to a very soft state. If swelling is prohibited the suction potential is maintained for unlimited periods of time, provided that the system is chemically and physically unchanged (no crystallization, no disintegration, no diagenesis). For the present concept this means that the opening of a rock joint in a deposition

hole filled with compacted bentonite will initiate an extrusion of swelling bentonite into the joint irrespectively of whether this happens during the deposition or 100 000 years later.

Practical consequences

The concept implies that the space between rock and compacted bentonite bodies and between these bodies and the canister is filled with loose Na bentonite powder which is applied and compacted by means of some ring-shaped device. Their width is assumed to be 3 cm and it is also assumed that the air-dry powder can be compacted to $\rho = 1.3 \text{ t/m}^3$ at $w = 10\%$.

When the deposition hole has been filled (the upper part is assumed to consist of a sand/bentonite mixture), a lid is bolted to a temporary concrete frame so that no water can enter the hole from above. This will be the condition until the tunnel is going to be filled.

However, water in rock joints flows towards the deposition hole and enters the outer space with the loose bentonite powder. The powder takes up water and its bulk density increases to about 1.75 t/m^3 . The water uptake in the loose powder can be assumed to be a fairly fast process to begin with when capillary forces largely contribute to the water transport in the non-saturated clay. As the degree of water saturation becomes high and swelling starts, the water transport is governed by the permeability which is very low (about 10^{-13} m/s). Thus, the completion of water uptake and swelling will probably take years or even decades. It should be mentioned also that if the confining rock has been sealed by bentonite injection and if ground water flow towards the deposition holes is reduced by means of galleries of fairly closely spaced wells (Fig. 8)

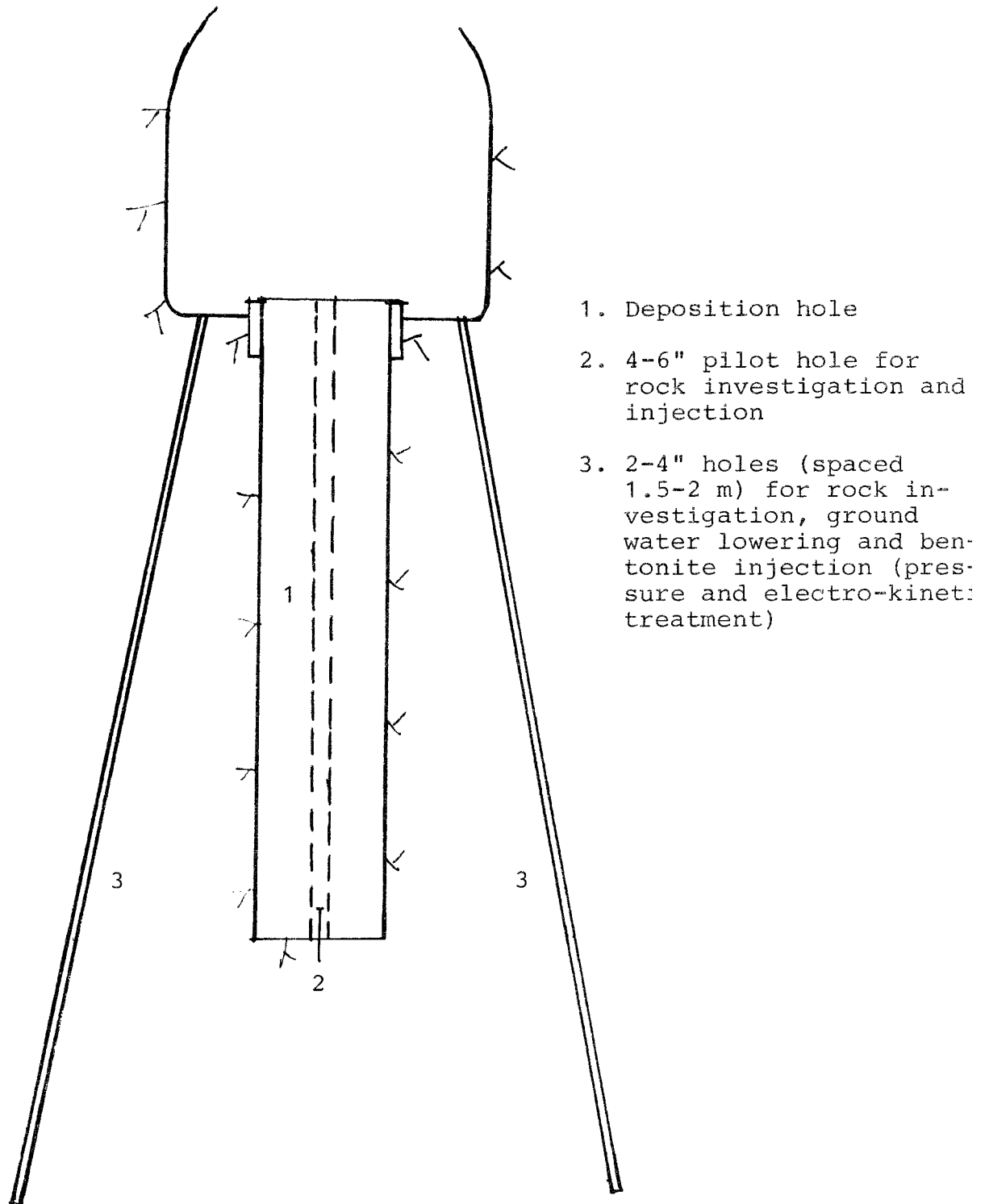


Fig. 8. Bore hole arrangement.

the water uptake will be much more delayed.

When the outer space with the loose bentonite is getting saturated, water is sucked into the highly compacted bentonite with a tremendous power. To begin with, the process is fast as in the outer space and the shallow parts of the bentonite bodies are soon water saturated and start to swell. Thereafter, further water transport towards the inner parts of the compacted bodies is increasingly governed by the permeability. Since the compacted bentonite bodies are much less permeable than the outer zone of loose bentonite (permeability ratio at least 1:10) this zone will be saturated before the water uptake in the compacted bodies has proceeded very far. Thus, water access to the compacted bodies will be uniform and a fairly uniform water uptake and swelling of the compacted bodies will therefore take place.

The joints between the compacted bentonite bodies will let water through more easily than the homogeneous bodies, so water will enter the inner zone of loose bentonite at a fairly early stage. It will be uniformly distributed in the inner zone from which it is then sucked into the compacted bentonite bodies. Again, this process will be uniform which means that the canister will not be subjected to local forces and bending momenta by non-uniform swelling.

As concerns the situation before water uptake, it should be noticed that neither the bentonite powder close to the canister nor the highly compacted bentonite bodies will fissure or crack due to the 80°C surface temperature of the canister. This is because the water content is much lower than the shrinkage limit. Thus, even if some water is evaporated the volume of the bentonite mass will stay constant. When water is finally brought into the

inner parts some of it will be evaporated while some is absorbed by the bentonite. Experience shows that the degree of water saturation will be high (>60%) also close to the 80°C canister periphery. The high water pressure produced after the closure of the deposition plant will dissolve all enclosed air and cause 100% water saturation. The time required to obtain complete water saturation and swelling of the compacted bentonite bodies is difficult to estimate. It will probably be some hundred or even thousand years. There are reasons to believe that water uptake leading to a fairly high degree of water saturation takes place after a much shorter time. Complete swelling, however, will take a very long time.

Development of swelling pressure in bore holes and tunnels

If the confining rock is sealed by bentonite injection and the ground water pressure is reduced by means of wells, the water flow through the rock to the deposition holes will probably be so minute that the majority of the water required to complete the swelling of the bentonite will be furnished by the tunnels after their closing and not by the confining rock. The water will enter the outer zone of loose bentonite which will be in contact with the lower bed of the tunnel fill in the absence of the lid and concrete frame.

The water uptake in the outer zone of loose bentonite in the deposition hole creates a swelling pressure of about 1-2 MPa when the water uptake is completed. During the period when the lid is still in place the pressure will probably not exceed 100-500 kPa for which the lid should be designed. Also, the concrete frame must be anchored in the rock to take the corresponding load.

Later, when the water uptake and swelling is complete

in the integrated bentonite complex in the confined deposition hole, the density of the bentonite will be uniform and about 2.15 t/m^3 . The swelling pressure exerted on the confining rock as well as on the canister will therefore be of the order of 8-10 MPa. Since the water uptake will not be completed until the lids have been removed the bentonite will not be perfectly confined during this process. In fact the pressure exerted by the expanding bentonite in the deposition hole will displace the tunnel fill. It will be shown later in this report, however, that this displacement is so small that it will only slightly affect the physical state of the bentonite. The mass will be very homogeneous without cracks, fissures or local density variations when the water uptake and swelling have finally taken place.

As concerns the tunnels, where the water uptake and swelling will probably be completed before this state is reached in the deposition holes, the swelling pressure will be very moderate. According to a preliminary investigation (JACOBSSON & PUSCH, 1977) this pressure will probably be of the order of 50-150 kPa.

Geotechnical properties of the water saturated bentonite (bearing capacity, settlement of canister)

The "air dry" bentonite body resting on the thin bed of bentonite powder at the bottom of the deposition hole, has to carry the weight of the canister and a number of bentonite bodies. In a later phase when the lids have been removed and the heavy tunnel fill is applied, the maximum vertical pressure on the lowest body may approach 1.0 MPa. This pressure does not produce fracture or significant deformation since the bentonite body has a perfect lateral support.

When water has been taken up and swelling has taken place the bearing capacity is changed. A very con-

servative way of expressing this capacity is to use the PRANDTL relationship for a load on the top of a weightless mass with the shear strength τ_f :

$$q = \lambda \cdot 5.14 \tau_f \quad (4)$$

where q = bearing capacity

λ = shape factor (1.3 for circular load)

τ_f = shear strength

It we apply for $\rho = 2.15 \text{ t/m}^3$ the value $\tau_f = 1500 \text{ kPa}$ (cf. Table 3) we obtain $q = 10 \text{ MPa}$. Since the pressure produced by the canister is only 0.4 MPa the factor of safety is obviously very high. It should be noticed that in this case the canister is the only load-producing unit in the bore hole. The weight of the bentonite mass and the overlying fill in fact improve the bearing capacity.

A slow settlement of the canister can take place since the water saturated, swollen bentonite certainly has viscous properties. An utterly conservative estimation concerning the settlement rate is made by assuming the canister to rest on a cylindrical bentonite column. For clay the following relationship is valid:

$$\log \dot{\epsilon} = C - m \log \frac{t}{t_1} \quad (5)$$

where $\dot{\epsilon}$ = axial creep rate

C = constant

m = parameter

t = time after load application, in minutes

t_1 = reference time, 1 minute

For a typical soft Quaternary clay with a shear strength of 20 kPa m can be taken as 0.8 and $C = \log 0.03$ at a shear stress corresponding to 70% of the shear

strength. This yields:

$$\frac{d}{dt} = \log \dot{\epsilon} = \log \left[0.03 \left(\frac{t}{t_1} \right)^{-0.8} \right] \quad (6)$$

$$\int_{\epsilon_2}^{\epsilon_2} d\epsilon = \int_{t_1}^{t_2} 0.03 \left(\frac{t}{t_1} \right)^{-0.8} dt \quad (7)$$

$$\epsilon = 0.15 (t^{0.2} - 1) \text{ in percent} \quad (8)$$

If the height of the column is 1 m the settlement will be 3 cm for $t = 10$ years, 5 cm for $t = 100$ years, 8 cm for $t = 1000$ years, 13 cm for $t = 10\,000$ years and 33 cm for $t = 10^6$ years.

Now, considering the fact that not only the column below the canister but also the rest of the bentonite mass contributes to the support, we can safely state that the settlement will only be a small fraction, 1/5 to 1/10, of the values mentioned. Furthermore, the viscosity of the bentonite is at least $10 \cdot 10^3$ times higher than that of the Quaternary clay. This means that the settlement will be insignificant.

Stress/strain effects

The consequences of water uptake and swelling are of great importance. There are two positive effects: swelling produces a pressure which extrudes dense bentonite out through any opening that might be formed in the confining rock. Also, the swelling pressure brings the bentonite mass to fill the deposition hole completely with a preserved density also if minor, local displacements should occur.

A number of possible negative effects must be considered, however. One could be that extruded bentonite may involve large losses of this substance or that extruding bentonite could have negative effects on the stress situation in the confining

rock. Another one is that swelling bentonite moves into the tunnel fill and displaces it. All these possibly negative effects will be considered in this chapter.

Extrusion of bentonite into rock joints

One of the advantages of using highly compacted bentonite in the deposition holes is that fissures and joints formed or opened in the confining rock tend to be sealed by bentonite which swells and is extruded through such openings. The matter is dealt with in detail in a report by PUSCH (1978).

Fig. 9 illustrates the extrusion of bentonite into an opened joint (from left to right). The density and therefore also the viscosity are both decreasing with increasing distance from the rock wall. The permeability, on the other hand, is changed in the opposite manner.

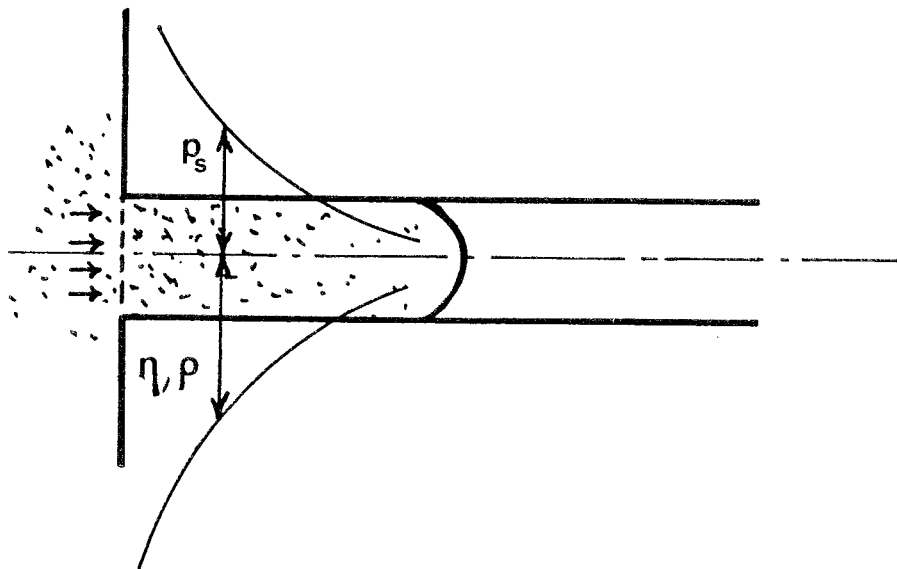


Fig. 9. Schematic picture of the influence of extrusion depth on swelling pressure (p_s), viscosity (η) and bulk density (ρ).

The driving force to bring water into the highly compacted bentonite and to extrude it through opening joints is the swelling power of the bentonite. The water transport rate and therefore the swelling rate are governed by the permeability and the hydraulic gradient.

It is obvious that the rate of water transport is difficult to estimate since not only the permeability but also the hydraulic gradient is a function of the extrusion depth. The gradient is due to the suction which decreases with decreasing density. This is also the case with the permeability.

In principle the rate of water uptake is a limiting factor as concerns the extrusion rate. If it were the only factor, the extrusion rate would be independent of the joint width, which is not the case as shown by a series of experiments (PUSCH, 1978). This means that for sufficiently narrow joints the wall friction and internal friction (viscosity) of the clay govern the rate of extrusion.

The flow of bentonite in plane joints with a large extension and a constant width can be described by applying POISEUILLE's law, assuming the pressure gradient to be caused by the reduction of the swelling pressure in the flow direction. It turns out that the calculated extrusion rate is in reasonable agreement with experimentally observed rates for joint widths equal to or smaller than about 1 mm. However, when the extrusion depth is one or a few decimetres the time for water to be transported through the extruded body and into the swelling compacted body becomes the governing factor for the extrusion rate which is then successively reduced.

The front part of the extruded, fairly dense bentonite body will be dispersed as a consequence of the

swelling process. It is transformed to a very dilute gel which moves further into the joint. The density of this gel is low, in its front part extremely low, but its permeability is still low which means that the water transport backwards into the denser bentonite mass with a higher swelling potential is slowed down. Very probably the swelling rate of this front part is higher than the rate of extrusion of the denser bentonite body. Although additional bentonite is slowly pressed into the joint the front of this denser body will probably soon be stationary because of the dispersion of its outer part.

Obviously, the width of joints or fissures formed or widened after the closure of the deposition plant must be considered before we can understand the effects of bentonite extrusion through them.

The basic question is: How and where can open joints or fissures be formed in the rock where deposition holes have been bored? The answer is that such openings are formed when tensile stresses develop. The problem is therefore to find out whether such stresses can be produced.

This discussion is confined to the case where the vertical and one of the horizontal stresses in the rock are equal. They are also assumed to be principal stresses. These assumptions are realistic for the sites where deposition plants are going to be located. The second horizontal stress, which is also a principle stress, is assumed to be $1/3$ of the first-mentioned horizontal stress. This is a rather common (non-favourable) situation which may be valid also for the deposition sites.

Case_I

Let us first assume that the tunnels are oriented parallel to the minor horizontal stress in the rock.

This means that the vertical primary stress is about 13 MPa and that the horizontal primary stress (perpendicular to the tunnel axes) is also 13 MPa.

The tangential stress, which is the interesting quantity, can be estimated by applying continuum mechanics. If the tunnel section is approximated to a circle the general expression for the tangential stress is:

$$\sigma_{\theta} = \frac{\sigma_x + \sigma_y}{2} \left(1 + \frac{a^2}{r^2}\right) - \frac{\sigma_x - \sigma_y}{2} \left(1 + \frac{3a^4}{r^4}\right) \cos 2\theta \quad (9)$$

where σ_x, σ_y = principal stresses
 a = tunnel radius
 r = distance from tunnel center to element in question
 θ = angle between x direction and radius vector to element in question
 Note: Compressive stresses are taken as positive.

We are interested only in the rock close to the periphery where the compressive stresses may be low. For $\sigma_x = \sigma_y = 13$ MPa and $r \sim a$ we obtain:

$$\sigma_{\theta} = 26 \text{ MPa} \quad (10)$$

This value means that all rock elements which constitute the tunnel periphery are compressed and equally stressed. The stress is easily tolerated by granite or gneiss.

If we now consider a deposition hole extending from the bottom of the tunnel, this hole will be situated in the stress field shown in Fig. 10. The situation may in fact be rather different due to a number of factors but the assumed conditions represent a conservative case for the part of the deposition hole

which is close to the tunnel base. σ_x drops fairly rapidly at increased depths below this base and is only slightly more than 13 MPa at the base of the deposition hole.

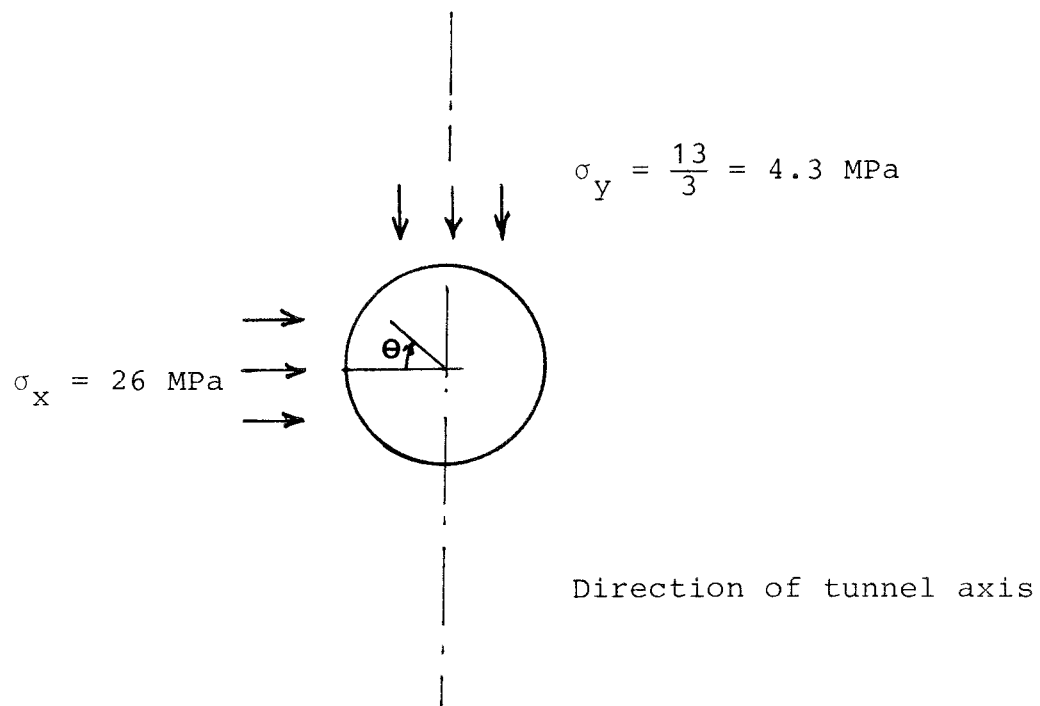


Fig. 10. Stress field in rock immediately below tunnel.

Now, applying again Eq. (9), we obtain for elements at the periphery of the deposition hole close to the tunnel base:

$$\sigma_{\theta} = 30.3 - 43.4 \cos 2\theta \quad (11)$$

This expression means that for a large θ -interval, corresponding to about one quarter of the hole periphery, there are tensile stresses in the surrounding rock. For elements at the periphery of the deposition hole at its base, on the other hand, there will hardly be any tensile stresses at all in the surrounding rock. The conclusion must be that vertical joints extending from the deposition holes tend to be opened or that new fissures may be developed by the excavation of the holes. Their width may extend from a fraction of a millimeter to one or two millimeters at maximum. The essential thing is that their extension cannot be more than a few meters because outside this range all stresses are compressive and of considerable magnitude. It is obvious, however, that sealing of such openings by injection is a necessary operation.

When the bentonite has been inserted and swelled, it exerts a pressure of as much as 10 MPa on the bore hole periphery and the dimensions of the hole will be somewhat increased. It expands more in the direction of the minimum primary stress (direction of tunnel axis) and this may, depending on POISSONS's ratio, increase the width of existing joints. The increased width of these joints (or of new fissures) will most certainly be less than a millimeter. Again, the essential thing is that their extension cannot be more than a few meters because of the stress situation outside this range. Consequently the risk of uncontrolled bentonite loss is none. Only a very small amount of bentonite will be redistributed from the deposition hole out into the confining rock.

Case II

If it is assumed that the tunnels are oriented parallel to the maximum horizontal stress, the vertical primary stress is about 13 MPa while the horizontal primary stress perpendicular to the tunnel axes is about 4.3 MPa.

The minimum tangential stress is obtained when $\cos 2\theta$ in Eq. (9) is maximum, which is the case for $\theta = 0$, that is at the top and bottom of the tunnel. Here the tangential stress is zero.

If we now consider a deposition hole from the bottom of the tunnel, this hole will then be situated in a stress field where $\sigma_x = 13$ MPa and $\sigma_y = 0$ close to the tunnel base. At the base of the deposition hole the situation will be as in the previously considered deposition hole.

By applying Eq. (9) we obtain for elements at the periphery of the deposition hole close to the tunnel base:

$$\sigma_{\theta} = 13 - 26 \cos 2\theta \quad (12)$$

As in Case I we find that tensile stresses develop over a large part of the hole periphery in this upper part of the hole while there will hardly be any tensile stresses at the base of the deposition hole. The same conclusions can therefore be drawn as concerns the extrusion and loss of bentonite as in Case I.

Some general conclusions can also be drawn as concerns the influence of the initial stress situation in the rock. Thus, it is obvious that there are great advantages in selecting for deposition purposes a

region where the primary stress situation is fairly isotropic and, hence, where tensile stresses will not be formed at all. If the horizontal stress field is markedly anisotropic the tunnels should preferably be oriented so that the primary stresses perpendicular and parallel to the tunnel axes are of similar order of magnitude.

The fact that the assumed stress situation does not produce tensile stresses in the deeper part of the deposition holes suggests that the holes should be somewhat deepened. It is therefore recommended that the final choice of tunnel orientation and depth of deposition holes is made after the rock investigation. Rock stress measurement is certainly an important part of this investigation.

The conclusion that only narrow joints or fissures will be formed can now be taken as a basis for the following statements:

- The rate of bentonite extrusion will be very slow except for the first few centimeter move which may take place in a few months.
- Even after several thousand years the extrusion of fairly dense bentonite will probably not exceed one or a few decimeters in such narrow joints.
- In the outer part of the extruded bentonite body there will be a successive transition to a very soft, dilute bentonite gel. An unexpectedly low ground water salinity may yield a gel/sol transition but the sol will then not consist of individual montmorillonite flakes but of fairly large aggregates which will be stuck where the joint width decreases. Here, such aggregates will collect, thus forming a dilute, but fairly impervious

and very surface-active fill.

These statements tell us a great deal about the influence of the extrusion depth on the swelling pressure. As shown by PUSCH (1978) the average swelling pressure of the extruded bentonite body can be estimated by means of Fig. 11.

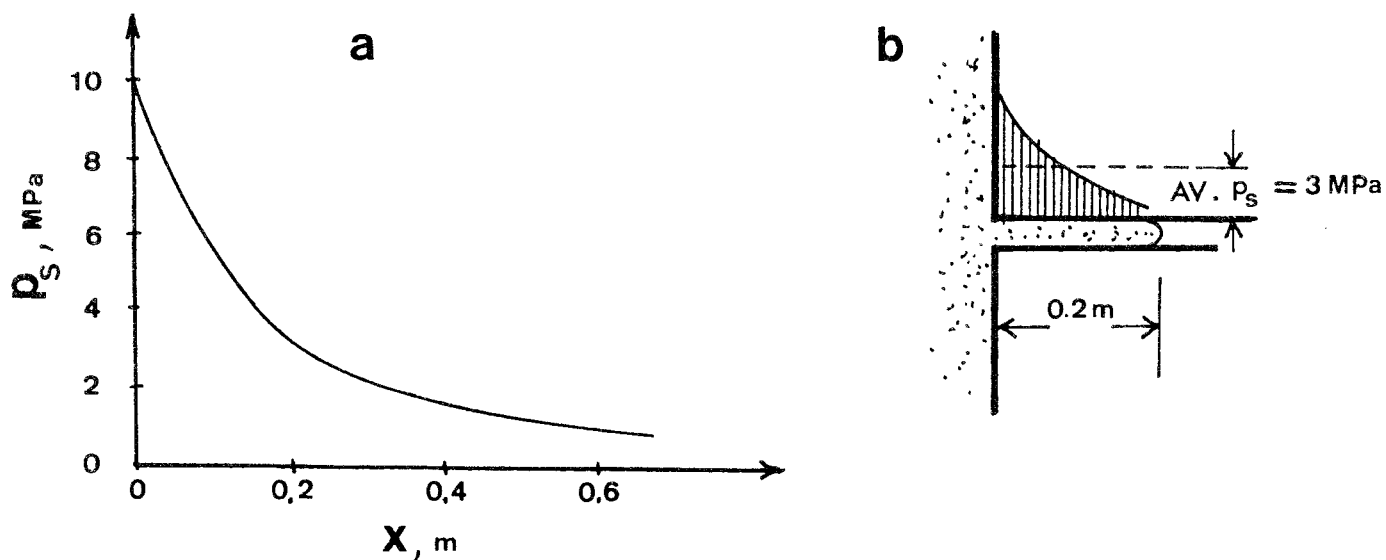


Fig. 11. a) Average swelling pressure p_s in extruded bentonite versus extrusion depth x .
b) Example for $x = 0.2$ m showing probable distribution of swelling pressure.

The swelling pressure given by Fig. 11 a is conservative (too high). This is because the method of calculation implies a uniform swelling pressure within the whole body while the real distribution must follow some power law with the maximum pressure at the rock wall.

The consequence of the swelling pressure on the

stress situation in the confining rock will be that it increases the tangential stress which tends to bring back the original shape of the hole section, i.e. the joints will not open. It is obvious, however, that this effect is very small since the swelling pressure will be insignificant already a couple of decimetres from the rock surface. It is important that the radial rock stress produced by the swelling pressure tends to reduce the width of a majority of joints.

Displacement of bentonite into tunnels

When the highly compacted bentonite in the deposition holes has taken up water and expanded to fill the holes it will exert a pressure of about 8-10 MPa on the overlying tunnel fill. The displacement caused by this pressure will produce a displacement of bentonite into the tunnel fill and a concomitant reduction of the density of the bentonite in the deposition holes. The displacement can be determined in several ways.

One way of estimating the displacement is to consider it as the classical oedometer case where a cylindrical body of tunnel fill is uniaxially compressed under the action of a circular load corresponding to the swelling force of the compacted bentonite. This is a very conservative approach. The standard equation for the axial compression is:

$$\delta = \frac{H}{m\beta} \left[\left(\frac{\sigma' + \sigma_0'}{\sigma_j} \right)^\beta - \left(\frac{\sigma_0'}{\sigma_j} \right)^\beta \right] \quad (13)$$

where δ = compression
 H = height of compressible body
 m, β = parameters
 σ' = applied effective pressure (swelling pressure)

$$\begin{aligned}\sigma_0' &= \text{average existing pressure (vertical} \\ &\quad \text{pressure at half height)} \\ \sigma_j &= \text{reference stress} = 100 \text{ kPa}\end{aligned}$$

Reasonable values are $\beta = 0.6$ and $m = 1000$ (according to JANBU's values for medium stiff to stiff sand/ /moraine) $H = 4$ m, $\sigma' = p_s = 10\,000$ kPa and $\sigma_0' = 30$ kPa. Using Eq. (13) we obtain $\delta \approx 0.1$ m.

Alternatively, by applying the theory of elasticity we obtain for a circular load on a semi-infinite elastic mass:

$$\delta = p_s \cdot 2r \cdot \frac{1-\nu^2}{E} \quad (14)$$

where p_s = swelling pressure (10 MPa)
 r = radius of deposition hole (0.75 m)
 ν = POISSON's ratio (0.3)
 E = modulus of elasticity, 150 MPa (empirical value for well compacted sand and gravel)

Eq. (14), which considerably exaggerates the displacement, also yields $\delta \approx 0.1$ m. Since it may be difficult to obtain a uniform compaction of the upper part of the tunnel fill it is reasonable to believe that the average E -value will be lower than 150 MPa. A safe assumption would be that δ approaches 0.2 m.

A displacement of 0.2 m involves an insignificant reduction of the average bentonite bulk density in the deposition hole. Thus, if the whole mass is affected, the displacement will reduce this density by about 2%. If only the bentonite above the canister is affected,¹⁾ its average bulk density will be reduced from 2.15 t/m³ to 2.00 t/m³ if the swelling pressure is maintained. Since the swelling will in fact reduce this pressure to about 4 MPa an equilibrium will probably be obtained at a displacement somewhere

¹⁾ This may be a condition for thousands of years because of the slow swelling rate.

between 0.1 and 0.2 m. It is therefore concluded that the displacement does not noticeably affect neither the swelling pressure nor the physical and geotechnical properties of the bentonite in the deposition hole.

It should be added that there will not be any intrusion of bentonite into the voids of the tunnel fill. This is obvious from geological evidence and from the self-injection experiments mentioned previously in this report. The maximum pore size of the tunnel fill is only a very small fraction of the joint width used in the experiments.

One possible effect of the displacement of bentonite into the tunnels could be that the wall friction in the upper part of the deposition hole would be so strong that the moving bentonite could shear off large rock pieces from the tunnel base (Fig. 12).

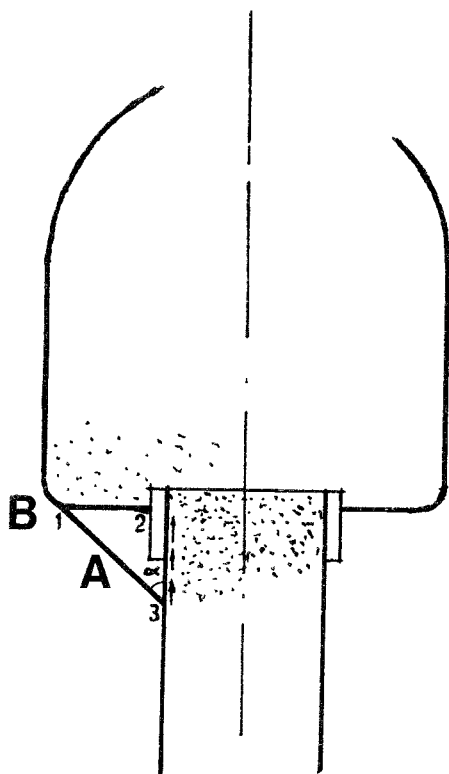


Fig. 12. Possible shear effect produced by displacement of bentonite.

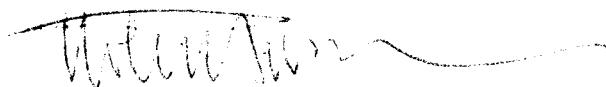
If we consider the counter-pressure from the fill and take into account the possible joint pattern in the rock which would permit such a shear process, we find two possible effects. One would be that an existing joint oriented as the A-line in Fig. 12 could open a few centimeters. The opening can proceed only to point B and not further off into the rock mass because of the large compressive stresses in this region. Such an opening will be rapidly filled by "self-injecting" bentonite with only minor density changes in the deposition hole. The other effect would be that the block (1,2,3) is moved along the A-joint without opening it. Also in this case the movement will be moderate with only minor effects on the bulk density and other physical properties of the bentonite mass. If the distance from the tunnel floor and the top of the canister is increased by 1 m (cf. Fig. 7) i.e. to 2.5 m, the negative effect of such shear processes is eliminated.

Finally, a special case which concerns gas production will be considered here. A probable effect of corrosion of the canister could be a production of hydrogen gas. It may be questioned whether this is possible at all since swelling bentonite will penetrate and fill any corrosion cavern and thus prevent the formation of open space which is required for rapid chemical reaction. However, it is assumed here that the gas is produced instantly and that it is not dissolved in the water or transferred to the surroundings by diffusion. If the reaction is complete it means that a gas amount of 220 m³ per canister is produced at normal air pressure. This corresponds to 4.5 m³ at 5 MPa gas pressure and 2.2 m³ at 10 MPa. The gas exit will of course be where the corrosion has punctured the canister. Capillary effects will prevent water from being extruded out of the clay pores by the gas. Instead, the gas will displace water-saturated clay. A gas-filled space will be formed close to the canister. Since minimum resistance

is exerted by the overlying tunnel fill, the gas void will tend to expand upwards thereby displacing the bentonite and the tunnel fill.

Applying the previous reasoning concerning the displacement of tunnel fill by mechanical pressure it can be concluded that the gas-filled space will be confined to 0.5-2 m³ adjacent to the canister. It will not be in contact with the rock or the tunnel fill and it will probably have a fairly regular and smooth contour. Its expansion, which will ultimately lead to an equilibrium, produces a rather local compaction of the tunnel fill above the deposition holes. It will not be a matter of extrusion of the fill out of the tunnel system.

Luleå 1978-02-25



Roland Pusch

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