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Coupled Processes in Geomechanics

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ABSTRACT

The safe operation of an underground repository for the disposal of nuclear wastes and the isolation of these wastes from the accessible environment, may be affected adversely by the formation of new fractures in the rock close to these excavations or by movements across pre-existing fractures. Both phenomena may so enhance the permeability of the rock as to allow possibly contaminated groundwater to reach the accessible environment. For hard rocks, the formation of new fractures in the regions of high stress concentration adjacent to excavations is analysed in terms of "breakouts" or "spalling". In rocksalt, creep may allow the stress adjacent to excavations to decrease below the hydrostatic pressure so that hydraulic fractures may form, or the extension of layers other than salt may produce new hydraulic conduits. The deformation of fractures in response to changes in stress pore pressure, and temperature are analyzed for pre-existing fractures, as is also the shear stability of the fractures. Experience and in situ testing are seen to be of the greatest importance in identifying and understanding coupled phenomenon in geomechanics.

INTRODUCTION

The overall geomechanical requirements for a waste repository are to ensure safety during repository excavation, waste emplacement and backfilling, and to obviate disturbances to the rock mass that could result in the release of radioactive material to the accessible environment after closure. To meet these requirements it is necessary to demonstrate that

- the deformation of canister holes will be acceptable and stable;
- the deformation of rooms will be acceptable and stable;
- any new rock fractures which may be produced will not lead to unacceptable seismicity or unacceptable increases in hydraulic, or decreases in thermal, conductivities;
- the opening and, perhaps, closing of pre-existing and, new joints and fractures will not lead to unacceptable increases in hydraulic conductivity;
- the shearing of pre-existing joints and fractures will not produce unacceptable seismicity or unacceptable increases in hydraulic conductivity;
- backfilling and sealing of holes, rooms andshafts will provide adequate containment of, possibly contaminated, groundwater.

HOLES AND ROOMS

Hard Rock

It is obvious that the excavation of boreholes, for canister emplacement, and drifts, for the repository access ways and rooms, will result in a redistribution of the rock stresses adjacent to these excavations. At the depths of most proposed repositories, say 500 m to 1000 m below surface, the vertical stress will result essentially from the weight of the overburden and will, therefore, have values between 12 MPa and 25 MPa. Horizontal stresses in the shallow crust have values that range from as little as a third of the value of the vertical stress to more than three times its value (Hoek and Brown, 1980). The uniaxial compressive strengths of rocks at proposed repository sites range from essentially zero for salt, up to 250 MPa or more for basalt or granite. These strengths are typical mean values from laboratory measurements on small specimens of rock. The statistical spread of values from individual specimens may be as great as 50 percent. Furthermore, the strengths of small samples are considered to be significantly greater than those of rock masses in situ (Goodman, 1980; Hoek and Brown, 1980). Accordingly, it is not improbable that the concentration of tangential stress in the wall of an excavation may be sufficient for the tangential stress to exceed the strength of the rock. Indeed, there is already clear evidence from potential repository sites that this will occur. For example, the walls of exploratory boreholes in the basalts at Hanford have been found to be "broken out" (Kim, et al, 1984), as have large-diameter northern at the Nevada Test Site, (Springer, et al, 1986).

Ordinarily, well bore breakouts and the analogous spalling of the walls of larger excavations would not be viewed as a phenomenon involving coupling. However, the maximum elastic stress concentration around openings depends upon the shape of the excavation. Specifically, the stress concentrations at the ends of the major and minor diameters parallel to the axes of the principal stresses for holes with elliptical cross sections are given by:

\[
\sigma_{xx} = \left[1 + 2 \left(\frac{a}{b}\right)\right] \sigma_y - \sigma_z \quad (1)
\]

\[
\sigma_{yy} = \left[1 + 2 \left(\frac{a}{b}\right)\right] \sigma_z - \sigma_y \quad (2)
\]

where \(\sigma_x\) and \(\sigma_y\) are the maximum and minimum principal stresses in the plane of the hole;

- \(a\) = semi axis of the elliptical hole in the \(x\)-direction,
- \(b\) = semi axis of the elliptical hole in the \(y\)-direction. (Jaeger and Cook, 1979).

In general, the maximum horizontal stress concentration around an opening in an elastic material increases approximately as the square root of the ratio between the distance of the hole surface from its center and the local radius of the curvature of that surface (Jaeger, 1979). The sketch in Figure 1 illustrates how the shapes of a circular borehole change as a result of "break out". A drift with a circular cross section "spalls" in much the same way. It can be seen that the change in cross section of the hole brought about by breakout or spalling accentuates the tangential stress concentration in the region adjacent to the breakout, causing further degradation. There is, thus, a powerful coupling between the geometry of a hole and its stability.

In practice, it would be very difficult to drill and keep open large numbers of boreholes for canister emplacement under conditions where breakout occurs. Likewise, it would be onerous and
Thermoelastic stresses can be expected to exacerbate spalling. The tangential stress around an excavation with a surface temperature \( T_s \) is increased by
\[
\Delta \sigma_\theta = \frac{\alpha E T_s}{(1-\nu)}
\]
where
\[
\alpha = \text{linear coefficient of thermal expansion}
\]
\( E = \text{Young's modulus} \)
\( \nu = \text{Poisson's ratio} \) (Timoshenko and Goodier, 1951).

If the interior surface of an excavation is at a constant temperature, breakouts and spalling can be analyzed as described above by substituting \( C_s' = C_s - \Delta \sigma_\theta \) for the uniaxial compressive strength, that is,
\[
S = \frac{(\sigma_z - \sigma_y)}{(C_s - \Delta \sigma_\theta)}
\]

Should thermally exacerbated spalling occur in a canister hole, an annulus of broken rock with diminished thermal conductivity will be created around the canister. This will lead to greater canister temperatures in order to maintain the flow of heat through the rock equal to that generated by radioactive decay in the canister. While this increase in temperature could prove damaging to a canister, the temperature of the rock at the new boundary of the solid rock will diminish as the reciprocal of the logarithm of the diameter of solid boundary. This decrease in temperature will reduce the thermally induced stress until spalling ceases. Alternatively, if the annular space between the canister and the hole is limited, this space will fill with spalled rock and the dilatation associated with further spalling will compress it, eventually generating a normal stress sufficient to prevent any new spalling from occurring.

This sequence of events was predicted and observed in one of the experiments at Stripa in which a full scale electrical heater with a power output of 5 kW simulated a canister of spent fuel 3 years out of the reactor, (Cook and Meyer, 1980). Figure 3. In this particular experiment, the rock mass in which the canister hole was drilled was heated also by a ring of peripheral heaters to simulate the thermal field in an actual repository that would result from adjacent canisters of waste.

The phenomenon of wellbore breakouts has attracted considerable attention recently as a method for determining stresses in rock at depth. Gough and Bell (1981, 1982) have analyzed breakouts in terms of elastic stress distributions and a Mohr-Coulomb failure criterion. Zoback et al. (1985) making similar assumptions, show that breakouts result from shear failure of the rock where the compressive stress concentration is greatest around the hole. However, Fairhurst and Cook (1966) observed that spalling of the walls of the drifts occurs by rock splitting parallel to the direction of maximum tangential compression, and used a fracture mechanics model for the growth of Griffith cracks in compression to explain their observations. Freudenthal (1977) on the basis of a shear dilatancy model suggests that extensive fracturing may occur adjacent to excavations in rock subject to compression. In hard rock, spalling is characterized by extensive planar fractures, sometimes with dimensions of meters, that break the rock adjacent to the walls of excavations into sheets with thicknesses of the order to centimeters. The fractures follow the direction of the maximum horizontal stress with remarkable fidelity. Horii and Nemati Nasser (1985) have developed a numerical fracture mechanics model for the propagation of Griffith cracks in compression which results in spalling adjacent to a free surface.

Breakouts and spalling could make repository construction difficult and hazardous and may make retrieval very difficult, but the effect of this phenomenon may be most significant in terms of repository performance. The rooms and access ways of a repository constitute a network of interconnected hydraulic conduits, which did not exist in the natural state prior to repository excavation. Even filling these excavations with an uncemented backfill will reduce
their hydraulic conductivity to a negligible extent. To ensure isolation of the wastes, these conduits will have to be interrupted at many points in the repository by the installation of cemented plugs and seals. If breakouts and spalling are dilatant phenomena, the cracks and fractures associated with them will produce permeable zones in the rock adjacent to breakouts and spalling. These permeable zones constitute pathways by which possibly contaminated groundwater can be pass seals and plugs.

The strengths of even hard rocks are time dependent and decrease also with temperature and in the presence of water. Therefore, even where breakouts and spalling are not a problem in the short term of repository construction, they may become so in the longer term. Tapponier and Brace (1978) observed that virtually all cracks induced in Westerly granite at 23°C, a strain rate of \(10^{-9} \text{s}^{-1}\) and confining stress of 50 MPa were extensible. Kraus (1979) showed that confining pressure strongly inhibits crack growth and time to failure in constant stress creep tests on dry Barre granite. For example, at a stress difference of 0.7% of the short term strength, the time to failure at a confining stress of 33 MPa was about 15 x 10^9 whereas at a confining pressure of 0.1 MPa the time to failure was only about 400 s. Friedman et al. (1979) showed that the strength of charcoal granodiorite decreased from about 350 MPa at room temperature to only 140 MPa at a temperature of 400°C at zero confining pressure but that at a confining pressure of 50 MPa the strength decreased from about 830 MPa to 540 MPa, over the same temperatures. All the evidence seems to show that over an extended period of time, say decades, breakout and spalling are likely to occur adjacent to repository excavations, where the confining stress is virtually zero. This may produce zones of enhanced hydraulic conductivity in the rock. The only effective way to inhibit breakout and spalling is through the early application of a confining stress, that is, this stress must be applied within hours, preferably minutes, of excavation to take advantage of the higher, short term strength of the rock.

Rock Salt

Rocksalt has negligible shear strength. Breakout and spalling are not expected to be major problems. Indeed, the creep deformation of rocksalt is expected ultimately to encapsulate the wastes completely. However, the closure of excavations may occur much more slowly than laboratory tests would lead one to expect. Steady state creep of rocksalt under isothermal, triaxial compression in laboratory tests is a power function of the stress difference.

\[
\dot{\varepsilon} = A (\sigma_1 - \sigma) / K
\]

(8)

where

- \(\dot{\varepsilon}\) = strain rate;
- \(A\) = constant that depends upon the temperature;
- \(\sigma_1\) = maximum principal stress;
- \(\sigma_0 = \sigma_3 = \sigma = \) confining pressure;
- \(K\) = normalising factor, say, the bulk modulus;
- \(n\) = a hardening exponent with a value of typically 4 or 5.

For multiaxial stress states, equation (8) can be written as

\[
\dot{\varepsilon}_{ij} = \frac{3}{2} A (\sigma_0 / K)^{n-1} S_{ij}
\]

(9)

where \(S_{ij}\) are the deviator stresses, that is,

\[
S_{11} = \frac{(2\sigma_0 - \sigma_1 - \sigma_2)}{3}
\]

\[
S_{22} = \frac{(2\sigma_0 - \sigma_2 - \sigma_1)}{3}
\]

\[
S_{33} = \frac{(2\sigma_0 - \sigma_3 - \sigma_1)}{3}
\]

and

\[
\sigma_0 = \left[ \frac{3(S_{11}^2 + S_{22}^2 + S_{33}^2)}{2} \right]^{1/2}
\]

(10)

Equation (9) can now be written conveniently as:

\[
\dot{\varepsilon}_{ij} = \frac{1}{2\eta} \sigma_0^{n-1} S_{ij}
\]

(11)

where

\[
\eta = \frac{K^n}{(3A)}
\]

Consider now the creep deformation of a long, cylindrical hole in plane strain subjected to lithostatic stresses at infinity. Let the radius of the hole be \(r_s\) and the radial displacement of this boundary be \(u_s\). When the creep strain is larger than the elastic strain, the deformation of the rocksalt will be almost incompressible, that is

\[
\dot{u}_s = \frac{u_s}{r_s}
\]

(12)

where \(u\) = the radial displacement at any radius \(r\) and the dot signifies the time derivative.

The stress equation of equilibrium in cylindrical coordinates is:

\[
\frac{\partial \sigma_r}{\partial r} = \frac{\sigma_r - \sigma_\theta}{r}, \quad \frac{\partial \sigma_\theta}{\partial \theta} = \frac{\sigma_\theta - \sigma_z}{r}, \quad \frac{\partial \sigma_z}{\partial z} = \frac{\sigma_z - \sigma_r}{r}
\]

and the radial, tangential and axial strain rates are, respectively,

\[
\dot{\varepsilon}_r = \frac{\dot{u}_s}{r_s} = \frac{\dot{u}_s}{r^2}, \quad \dot{\varepsilon}_\theta = \frac{\dot{u}_s}{r_s} = \frac{\dot{u}_s}{r^2}, \quad \dot{\varepsilon}_z = 0
\]

(13)

so that, from equations (11) and (14) and (15)

\[
(\sigma_r - \sigma_\theta) = 2\eta \left( \varepsilon_r - \varepsilon_\theta \right)
\]

\[
= -4\eta \left( \frac{\dot{u}_s}{r_s} \right) \left( 3\eta \dot{\varepsilon}_s \right)^{1-n} / n
\]

where

\[
\dot{\varepsilon}_s = \left( \frac{2 \varepsilon_{ij} \varepsilon_{ij}}{3} \right) = \left( \sqrt{\frac{\pi}{3}} \frac{u_s}{r_s} \right) / r^2
\]

and, from equation (13),

\[
\sigma_r = \int \left( \frac{(u_s - \sigma_r)}{r} \right) dr
\]

\[
= 4\eta \left( \frac{\dot{u}_s}{r^2} \right) \left( 3\eta \sqrt{\frac{\pi}{3}} \frac{u_s}{r_s} \right)^{1-n} / n
\]

\[
= (3\eta \dot{u}_s r_s)^{1/n} \left( \frac{A}{3} \right)^{(1+n)/2} \int r^{(-2-n)/n} dr
\]

\[
= -(3\eta)^{1/n} \left( \frac{A}{3} \right)^{(1+n)/2a} \left( \frac{n}{2} \right) \frac{r^{-2/n}}{n} + C
\]

At \(r = \infty, \sigma_r = \sigma_H\) the lithostatic stress, so that

\[
\sigma_r = \sigma_H - (3\eta)^{1/n} \left( \frac{\dot{u}_s}{r_s} \right)^{1/n}
\]

(18)
and at \( r = r_s \), \( \sigma_r = 0 \), so that,
\[
\sigma_H = (3 \eta)^{1/n} \left( \frac{4}{3} \right)^{(1 + n)/2n} \left( \frac{n}{2} \right)^{-2/n} r_s \]
that is,
\[
\dot{\varepsilon} = \frac{(2 \sigma_H / n)^n}{|3 \eta^{(4/3)(1 + n)/2n}|} \]
but \( \dot{\varepsilon} / r_s = \dot{\varepsilon}_r \) and, comparing equation (19) with equation (8), it can be seen that the effect of the axisymmetric geometry is to diminish the effective magnitude of the stress difference reducing creep by a factor of \((2/n)^n\).

The tangential stress can be found from equations (13) and (18), and is given by,
\[
\sigma_\theta = (3 \eta)^{1/n} \left( \frac{4}{3} \right)^{(1 + n)/2n} \left( \frac{n}{2} \right)^{-2/n} r_s \]
Finally, the axial stress can be found from equations (9) (19) and (20). The axial strain rate \( \varepsilon_\theta = 0 \), \( \sigma_\theta \) cannot be zero so that, from equation (9) the deviator stress \( S_{33} \) must be zero, that is,
\[
(2 \sigma_\varepsilon - \sigma_r - \sigma_\theta) = 0
\]
or
\[
\sigma_\varepsilon = (3 \eta)^{1/n} \left( \frac{4}{3} \right)^{(1 + n)/2n} \left( \frac{1}{2} - \frac{n}{2} \right)^{-2/n} r_s \]
It follows that at \( r = r_s \)
\[
\sigma_r = 0
\]
\[
\sigma_\theta = \frac{2 \sigma_H}{n}
\]
\[
\sigma_\varepsilon = \frac{\sigma_H}{n}
\]
Equations (22) show that the tangential and axial stresses around a circular excavation in rocksalt become less than the hydrostatic stress if \( n \geq (\rho / \rho_w) \) where \( \rho \) = the mean density of the overburden and \( \rho_w \) is the density of water or brine.

In reality, repositories will not be isothermal so that \( \eta \), which is a function of temperature, will become a function of \( r \). This will affect the preceding result quantitatively, but not qualitatively, except to diminish the stresses adjacent to the excavation to an even greater extent when the highest temperatures correspond to the smallest radii.

The creep deformation of rocksalt produces large extensive strains in a radial direction. Many rocksalt deposits are interbedded with layers of shale, clay and anhydrite. Most of these layers are much more brittle than rocksalt so that they may be broken intensively by extension.

Consider a layer of brittle rock, embedded in rocksalt, at a height \( h \) above an excavation with a circular cross section. If \( u_s \) is the radial displacement at the surface of this hole with radius \( r_s \), then the radial displacement for incompressible flow at any other radius is given by:
\[
u = \frac{u_s \cdot r_s}{r}
\]
and the displacement parallel to any layer at \( y = h \) is
\[
\delta = \frac{u_s \cdot r_s \cos \theta}{r} \]
\[
\delta = \frac{u_s \cdot r_s \cdot x}{(x^2 + h^2)^{3/2}}
\]
The strain parallel to the layer is, therefore,
\[
\varepsilon_s = \frac{\partial \delta}{\partial x} = \frac{u_s \cdot r_s \left( h^2 - x^2 \right)}{(x^2 + h^2)^{3/2}}
\]
so that extension of the layer occurs for all \( x > h \). Brittle layers easily break in extension. The resulting void space may, therefore, increase their hydraulic conductivity significantly.

**FRACTURES**

**Fracture Aperture and Hydraulic Conductivity**

In rocks of intrinsically low permeability, joints, faults and fractures constitute the principal conduits for the transport of groundwater. Essentially, these fractures, collectively referred to as fractures, arise from the fact that their opposing surfaces are not completely flat, so that contact occurs between the surfaces across the asperities and voids exist between the surfaces elsewhere. The geometry of these contacts and voids varies with the stress normal to the fracture plane and the pore fluid pressure within it. The hydraulic conductivity of fractures depends upon the apertures of the voids and the manner in which they are interconnected. Under some conditions, shear displacements between the two surfaces may be expected to result in major changes in the geometry of contacts and voids.

It has been shown that, when the contact areas comprise a small fraction of the fracture surface area, fluid flow in a fracture is similar to that between parallel plates and the hydraulic conductivity of the fracture increases as the cube of the mean void aperture, (Iwai, 1976; Gangi, 1978). Experiments (Goodman, 1976; Bandis et al., 1983) and theory (Walsh, 1981; Brown and Scholz, 1985) based on the topography of the fracture surfaces show that this mean void aperture is a non linear function of the effective normal stress across the joint, that is, the difference between the total normal stress and the pore fluid pressure. As the normal stress increases so do the areas of individual contacts as well as the numbers of contacts. In principle, the hydraulic conductivity of a fracture can be derived from the mean joint aperture using the cubic relationship between these quantities.

However, experiments by Iwai (1976), Engelder and Scholz (1981) and Raven and Gale (1985) have shown that the cubic relationship between fracture aperture and hydraulic conductivity breaks down as high stresses and small apertures, Figure 3. For apertures of less than about 10\( \mu \)m increases in the effective stress to more than 10 MPa or 20 MPa, reduce the fracture aperture with little or no effect on the fluid flow. This means that under these conditions major void spaces in the fracture must close with virtually no change in the hydraulic resistance of the fracture. This can occur only if major void spaces are either isolated from the flow path or if the principal resistances to fluid flow occur in restricted connections between these voids. It follows that the hydraulic resistance of such restrictions must be comparatively insensitive to the normal stress, which would be the case if these restricted pathways resembled little pipes or tubes.

Actual curves showing the deformation of a fracture in basalt (Tsang and Witherspoon, 1981) and one in granite (Sun et al., 1985) are shown in Figure 4. Note that these curves are fitted quite well by the hyperbolic relationship proposed by Bandis et al., (1983).
where

\[ D = \text{volumetric closure of the fracture}; \]
\[ D_m = \text{maximum volumetric closure of the fracture}; \]
\[ K_i = \frac{\partial \sigma}{\partial D} \text{ at } \sigma = 0 \text{ that is, the initial specific stiffness of the fracture}. \]

The specific stiffness at any stress, \( K \), can in principle be measured seismically (Chen, et al., 1985), is given by

\[ K = \frac{K_i}{(1 - D/D_m)^2} \] (27)

Consider first how a change in the pore pressure, \( \Delta p \), at constant applied stress, \( \sigma \), changes the aperture of a fracture and hence its hydraulic conductivity. The aperture of a fracture depends virtually on the effective stress so that the aperture changes to correspond to a new effective stress \((\sigma - \Delta p)\). Of course, the reduction in effective stress reduces the frictional resistance to sliding between the fracture surfaces without changing the shear stress, so that the fracture may become unstable or less stable.

Second, consider a change in the pore pressure, \( \Delta p \), when the total displacement of the fracture and the adjacent solid rock is held constant, as in plane strain. Let the change in pore pressure occur first. This would result in dilatation of the fracture and an equal displacement between any two planes parallel to the fracture in the solid rock adjacent to the fracture. Next change the applied stress by an amount \( \Delta \sigma \). This will partially close the fracture and compress the solid rock one each side of the fracture by an amount \( \Delta a L/E' \) where \( L \) is the spacing between fractures, or the amount of solid rock on each side of the fracture, and \( E' \) is the plane strain modulus, that is,

\[ E' = E \frac{(1 - \nu)}{[(1 + \nu)(1 - 2\nu)]} \]

The correct value of \( \Delta \sigma \) is that which produces compression of the solid rock one each side of the fracture that equals exactly the change in fracture aperture brought about by \((\Delta \sigma - \Delta p)\), as is illustrated in Figure 5.

The effective normal stress across fractures is expected to change as a result of the redistribution of stresses around excavations, thermally induced stresses and pore pressures.

Effect of stress

In general, the excavation of openings always results in a decrease in the stress radial to the walls of the excavation. The stress in a tangential direction may increase or decrease depending upon the shape of the excavation and the ratios of the far field stresses. A criterion that the tangential stress should always exceed the original stress in any direction is most difficult to satisfy for a tangential stress parallel to the original maximum stress, that is,

\[ \sigma_T \geq \sigma_2 \]

If \( a \) is the major semi axis of an excavation with an elliptical cross section then

\[ \left[ 1 + 2\left(\frac{b}{a}\right) \right] \sigma_2 - \sigma_1 \geq \sigma_2 \]

or

\[ \frac{b}{a} \geq \frac{R}{2} \]

For maximum stability, \( b/a = 1/2 \), so that the maximum ratio of the stresses that satisfies both the stability and tangential stress criteria is \( R = \sqrt{2} \).

One of the experiments at Stripa (Wilson et al., 1983) involved the measurement of the flow of groundwater into a drift with a cross section of 5 m square and a length of 30 m, and of the pressure gradients in the rock out to radial distances of up to 40 m. Plots of the hydraulic head in the rock around this drift at different radial distances against the logarithm of distance, Figure 6, were linear, as would be expected. However, the lines fitting these data do not intersect the wall of the drift at zero head, indicating that the head loss through an annulus of rock about 2 m thick around the drift was relatively much greater than that in the rock further away from the drift. Whereas the hydraulic gradient between radial distances of 4 m to 40 m showed the permeability of the rock mass around the drift to be about \( 10^{10} \text{ m/s} \), the hydraulic gradient through the annulus showed the permeability of the rock immediately around the drift to be about \( 3 \times 10^{11} \text{ m/s} \). This reduction in permeability could easily be the result of the closure of fracture apertures by the tangential stress concentration in the rock adjacent to the drift.

Measurements of the stresses in the rock at Stripa indicated that the vertical stress was only about 5 MPa whereas the two horizontal stresses were about 20 MPa (Doe et al., 1981). This would result in the tangential stress in the roof and the floor of the drift being considerably greater than the original rock stress while that in the walls would actually be in tension. Very careful mapping showed that three of the four principal sets of joints dipped at more than 50° while the forth set was sub horizontal (Gale et al., 1982). The near vertical joints would have been closed under the high tangential stress which may account for the observed decrease in flow.

Effects of temperature

Increases in the temperature of the rock around a repository will produce thermal expansions that result in changes in stress. In general, within the volume of rock through which temperatures increase there will be an increase in compressive stress. In order to maintain overall equilibrium, it follows that outside the heated volume there must be exactly compensatory decreases in compressive stress. While increases in stress will tend to close fractures and therefore decrease the permeability of the volume of heated rock, the corresponding decreases in stress will enhance permeability outside the heated volume. In total, the net effect of changes in temperature must be to increase the overall permeability of the rock mass, because of the highly non linear relationship between the hydraulic conductivity of a fracture and the normal stress across it.

Porous materials have the same coefficient of linear expansion as the material of their solid matrix. However anomalously low values for thermally induced displacements and stresses were measured in the Stripa heater experiments (Chan et al., 1980). These values have been shown (Cook, 1983) to result from the deformation across fractures. Imagine a column of rock of length \( L \) that is subject to stress, \( \sigma \), contains a fracture normal to its axis, and is constrained at its ends from expansion, so that an increase in temperature produces also an increase in the stress across the fracture. Let \( \Delta \sigma^T \) be the mean increase in temperature of a heated length \( L \) of this column. In the absence of the fracture, the thermally induced increment of stress would be

\[ \Delta \sigma = \frac{l}{L} \Delta \sigma^T \] (26)

where

\[ \sigma = \text{coefficient of linear thermal expansion of the intact rock}; \]
\[ E = \text{Young's modulus}. \]

In the presence of the fracture with a specific stiffness \( K \) at a stress \( \sigma \), the thermal expansion would partly be taken up by closure of the fracture aperture so that the increment in stress would be

\[ \Delta \sigma' = \frac{1}{K} \frac{L + 1}{E} \Delta \sigma^T \]
The thermally induced displacements of any points on each side of the fracture are shown in Figure 7 as are the corresponding displacements that would exist in the absence of a fracture. From this figure it can be seen that the thermally induced displacements are altered so much by the fracture that no predictions about stress or displacements can be made without at least knowing the specific stiffness of the fracture and its location relative to the temperature field and the measuring points.

Effect of pore pressure

Probably the largest change in stress across a fracture will result from changes in pore pressure that can be expected as a consequence of the thermal expansion of water. For some period after excavation and before closure, the repository excavations will act as an hydraulic sink so that pore pressures in the rock around a repository will decrease from their original values, and this decrease will result in further closure of most fractures. Most repositories are expected to experience a thermal pulse as a result of the radioactive decay of the wastes. In many cases the peak temperature of this pulse will be reached within a few decades of waste emplacement (Wang et al., 1980). After the repository excavations have been closed, any repository below the water table will begin to resaturate. Some idea of the order of magnitude of this resaturation time can be gained from the large scale permeability experiment at Stripa. In this experiment the total inflow of groundwater into a drift measuring 30 m in length by 5 m square cross section, was 50 ml/min. Most repositories are expected to be placed in host formations with permeabilities no greater than the Stripa quartz monzonite. The time it would have taken to resaturate the drift at Stripa is almost $10^7$ seconds or about 30 years. If the resaturation period is greater than the time taken to reach peak temperature of the thermal pulse, thermal expansion of the water is not likely to pose any problem. However, if the resaturation period is less than the time taken to reach peak temperature, considerable increases in pore pressure may occur as a result of the relatively great coefficient of volumetric thermal expansion of water, about 0.0006/C. To estimate the change in pore pressure it is necessary to balance the thermal increase in the volume of water against the outflow. Outflow will always occur at least as readily as would steady one dimensional flow from the repository to the surface. If the permeability of the strata overlying the repository is $k$, the depth below surface is $H$, and the mean thickness of all the water in the excavations averaged over the plan area of the repository is $h$, and the rate of heating is $T$, then the pore pressure increment corresponding to steady flow could be found from

$$h = T = k \left( \frac{\Delta p}{H} \right),$$

that is,

$$\Delta p = \frac{H}{k} h a T$$

For, say $H = 1000$ m, $h = 1$ m, $k = 10^{-11}$ m/s, $a = 0.0006$C and $T = 3 \times 10^9$C/S (1C/year), $\Delta p = 1800$ m or 17.5 MPa.

This is not insignificant in relation to the expected values of rock stress.

Stability

Finally, it is necessary to consider the shear stability of fractures subjected to changes in stress, pore pressure, fluid flow and temperature, partly as potential sources of seismicity but mainly because it has been shown (Barton, et al., 1985) that relative shear displacements between fracture surfaces can enhance the hydraulic conductivity of fractures by a few orders of magnitude.

Jaeger and Cook (1979), and Cook (1981), Rice (1983) and Li and Rice (1983) have used two concepts of a complete stress strain curve, or load displacement curve, and an unloading modulus to analyze the stability of rock fractures.

Essentially, for any rock fracture some relationship must exist between the relative displacement between the two surfaces of the fracture and the shear stresses that arise from the resistance to sliding of these surfaces. This relationship specifies the complete strength of the fracture over all possible values of the relative displacement between its surfaces. (Conventionally, the “strength” is the maximum ordinate of this complete relationship.) We shall refer to this complete displacement-strength relationship as a reduced Griffith locus. In general, reduced Griffith loci are non linear and reflect the properties of the material and the changing geometry of the fracture.

Correspondingly, another relationship must exist between the shear stress applied to a fracture by the surrounding rock and the displacements through which the shear stress moves. This relationship specifies the stiffness of the loading system that applies the shear stress. (In conventional engineering “dead weight” loads, the magnitudes of which do not change with displacement, that is, their stiffness is zero, are often used: in this case only the “strength” is needed to determine stability.) The system may be the surrounding rock mass or a testing machine. We shall refer to this relationship as the unloading modulus. For loading systems in which the fracture is the only non linearity, unloading moduli are linear. Unloading moduli may be evaluated by applying hypothetical shear displacements to an idealized and frictionless fracture surface and calculating the shear stresses corresponding to each displacement.

An hypothetical reduced Griffith locus for a shear fracture, showing the shear resistance to sliding of the fracture as a function of the shear displacement between the two surfaces is illustrated in Figure 8. This locus represents the complete behavior of the fracture: displacements and stress move along the locus as the fracture grows. Every point on the locus corresponds to some limiting state of equilibrium on the fracture.

A fracture is embebed in rock which essentially behaves elastically. If, as described above, the fracture were replaced by a frictionless system through which the displacements were controlled, the unloading modulus of the rock mass could be determined by calculating the change in shear stress as a function of shear displacement. The slope of this unloading modulus is independent of the magnitudes of the stresses and displacements and of the fracture surfaces; it depends only upon the geometry of the system and the elastic constants of the rock mass.

Consider the stability of a fracture embedded in a rock mass with a given unloading modulus as the fracture evolves along a reduced Griffith locus. Let the shear stress in the rock mass be increased until a limiting state of stress exists at A. The unloading modulus at this point lies on a flatter slope than the reduced Griffith modulus, so that the fracture growth would be unstable and the applied load would move along the unloading modulus until the shear resistance of the fracture equals the applied shear stress at point B. In fact the energy supplied by the system as it unloads is greater than that absorbed by the fracture as it evolves from A to B in the amount shown by the area ABC, so that the fracture would shoot past the equilibrium position B to some other position such as D. At D the system is again stable and the fracture cannot evolve unless the shear stress increases to point E. Here again, an equilibrium state of stress exists and the fracture remains stable because the unloading modulus lies within the elastic part of the locus. Therefore, the fracture will evolve only if further shear displacement is provided to the system from outside, so as to bring the shear stress and displacements back on to the locus. Although the displacements will increase the stresses will decrease, because the fracture is evolving to a weaker condition. Such stable deformation can continue until the unloading modulus becomes tangent to the reduced Griffith locus at F where the system becomes intrinsically unstable, because the unloading modulus lies completely outside the Griffith locus.
Conceptually, the reduced Griffith locus can be found as follows. A fracture comprising any given geometry of asperities can be deformed elastically until the stress intensity factors at the asperities reach critical values. At this point further deformation would be inelastic. However, this end point of the elastic deformation defines a point on the locus corresponding to asperities of a particular size. As a result of inelastic deformation, or fracture growth, the size of the asperities will diminish. Other points on the locus can be found by determining the end points of elastic loadings for asperities of successively diminished sizes.

The effects of changing temperature on the stability of a fracture can be found, in principle, by using critical stress intensity factors that are functions of temperature. A higher temperature may lead to lower stress intensity factors and hence a smaller Griffith locus as illustrated by the dashed curve in Figure 8. Similar corrections can be made for the effects of fluid chemistry on the fracture energy or stress intensity factors.

Pressure solution at points of high stress concentration may be a relatively important phenomenon in determining fracture stability. The formation of stylolites has been analyzed in terms of pressure solution at points of high stress concentrations [Fletcher and Pollard, 1981; Merino et al., 1983]. The asperities of contact in a fracture constitute points of high stress concentrations at which pressure solution may be expected. This will have the effect of diminishing the size of the asperities in a manner similar to that in which their size is diminished by additional deformation. Therefore, pressure solution would be equivalent to increasing deformation; it may move a fracture from a point such as B to one such as E. At any point on the locus the instantaneous stability would still be determined by the slope of the unloading modulus compared with that of the locus at that point.

CONCLUSION

In this paper I attempted to identify coupled processes in geomechanics, that may be detrimental to repositories, by examining specific changes to the rock around canister holes and storage rooms that could adversely affect the use or the performance of a repository. It appears that the formation of new fractures in zones of the greatest stress concentrations adjacent to excavations, or the mobilization of old fractures, perhaps at quite large distances from the excavations, may generate new conduits or increase the conductivity of old conduits, so that, possibly contaminated, groundwater may reach the accessible environment sooner than is acceptable.

The approach used in this paper led to the analysis of specific potential problems in a repository namely, breakout and spalling in hard rocks and creep in rocksalt around holes and drifts, and fracture closing and opening and stability in the rock mass, rather than the identification of a variety of related phenomena, as was done in the panel report (Tsang and Mangold, 1984). Interestingly, both the approach in this paper and that in the report led to the identification of processes on the basis of relevant past experience rather than on the basis of fundamental scientific prediction. Certainly, well bore breakouts and the spalling of drifts as well as the still incompletely understood, complex mechanical, hydraulic and thermal behavior of fracture are more in the nature of discoveries rather than predictions. The seminal role of the Stripa experiments is obvious. I recognize that the emphasis in this paper has been on processes involving essentially mechanical hydraulic and thermal coupling, and that chemical effects may be more important than is recognized here. To some extent this is because relatively little work has been done on problems involving mechanical and chemical effects.

I am forced to conclude that there is no alternative to comprehensive in situ testing, both generic and site specific, to identify and resolve problems relating to waste repository performance in general and coupled processes in particular.

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**Fig. 1.** Regions of stability for excavations with circular, elliptical, and breakout cross sections in terms of the ratio of the principal stresses, R, and the ratio of the difference of the principal stresses to the uniaxial compressive strength, S.
Fig. 2. Axial sections through a canister hole containing a 5 kW heater at different times after emplacement, illustrating the thermal decrepitation of the rock adjacent to the hole. Inserts are sketches of the hole wall made using a borescope.

Fig. 3. Relationships between the mean aperture of fractures in rock and specific hydraulic flow through them, after Iwai (1976), Raven and Gale (1985), and Engelder and Scholtes (1981). Note that the cubic relationship between aperture and flow, a straight line in a semi-log plot, breaks down for small apertures.

Fig. 4. Experimental relationship between normal stress and fracture aperture in basalt and granite, after Tsang and Witherspoon (1981) and Sun et al., (1985).
Fig. 5. Changes in fracture aperture brought about by a change in pore pressure, $\Delta p$, under constant stress and constant strain conditions.

Fig. 6. Regression lines fitted to measurements of pressures at different radial distances for around a drift and the weighted average of these data, showing that the high head loss between the drift well and a radius of about 4 m.

Fig. 7. Sketches illustrating thermally induced displacements in a column of rock with and without a fracture.

Fig. 8. Sketch showing a reduced Griffith locus and an unloading line, indicating stable (BEF) and unstable (AB and F) shear displacements. The dashed lines indicate reductions in strength caused by temperature or chemistry.