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Äspö Hard Rock Laboratory

Backfill Requirements in KBS-type repository

A POSIVA/SKB Workshop

Held at the Äspö Hard Rock Laboratory, Sweden

August 27-28 2001

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Keywords: Backfill, KBS-3, ED2 repository, Friedland Clay, MX-80, bentonite compressibility

Abstract

A POSIVA/ SKB Workshop on the backfill requirements in a KBS-type repository was held August 27-28 2001 at the Aspo Hard Rock Laboratory, Sweden.

It was concluded that the backfilling and sealing arrangements for a KBS-3 repository aim to re-establishment of undisturbed conditions in the geosphere. The main function is to prevent the disposal tunnels and access routes into the repository from becoming major conductors of groundwater and transport pathways of contaminants, and to block inadvertent intrusion into the repository. The backfill should have chemically and mechanically favourable and predictable properties over long time periods and should not have any properties that could significantly impair the function of the other barriers.

The workshop included 10 presentations in three sessions as follows:

- The Role of Backfill in Safety Analyses.
- Backfill operations and techniques for application.
- Rock/Backfill interaction.

Sammanfattning

Ett gemensamt Posiva/SKB workshop om krav på återfyllning i ett KBS-3 förvar hölls på Äspö 27 – 28 augusti 2001.

Det konstaterades att återfyllnings- och förslutningsåtgärder i ett KBS-3 förvar syftar till att återställa ostörda förhållanden i geosfären. Åtgärderna ska förhindra att förvarstunnlar och tillfartsvägar blir hydrauliska flödesvägar för grundvatten och föroreningar, samt förhindra oönskat intrång i djupförvaret. Återfyllningen ska ha gynnsamma mekaniska och kemiska egenskaper, som är prognostiserbara över lång tid, och ska inte påverka egenskaper hos andra barriärer.

Mötet omfattade 10 presentationer inom följande teman:

- Betydelsen av återfyllningen i säkerhetsanalysen.
- Utförande av återfyllning.
- Samverkan berg/återfyllning.

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1 Introduction

Summary notes by Charles Fairhurst

Rock at depth is subjected to gravitational and tectonic forces. Removal of the rock to create the excavations necessary for a geological repository disturbs these forces and leads inevitably to a redistribution of stresses and deformation. The changes are concentrated in the immediate vicinity of the excavation. Depending on the magnitude of the pre-existing forces and the strength of the rock the redistribution may result in fracturing around the opening. The region within which such fracturing occurs is referred to as the excavation damaged zone or EDZ.

The crystalline rocks being considered for repository sites in Sweden and Finland are also permeable and saturated with water at a pressure that increases with depth. These rocks also contain occasional water-conducting fracture systems. Introduction of the excavation disturbs the groundwater pressure distribution, creating a pressure 'sink' that results in water flow into the excavation. The excavations provide a potential 'short-circuit' within a part of the regional groundwater flow system, particularly during the constructional and operational phases. Since a primary aim of the geological isolation of nuclear waste is to minimise transport of toxic radionuclides from the repository to the biosphere, it is necessary to eliminate, or greatly reduce, the possibility of flow along excavations once the waste has been placed underground.

Flow within the excavation itself can be suppressed by filling it "completely" with material of low hydraulic conductivity - 'backfill'. Flow through the EDZ requires additional measures, such as excavation of the damage zone locally and introduction of a low hydraulic conductivity seal (this requires careful planning to avoid extension of the EDZ behind the seal). The aim is to interrupt the continuity of the EDZ along the length of the excavation.

Thus, as noted by Autio et al. (paper 3, page 3):

The backfilling and sealing arrangements aim to re-establishment of undisturbed conditions in the geosphere (Posiva 2000). The main function is to prevent the disposal tunnels and access routes into the repository from becoming major conductors of groundwater and transport pathways of contaminants, and to block inadvertent intrusion into the repository. In a KBS-3 repository the functions of the backfill in the upper part of the deposition hole and in the tunnel further include

- *to keep the buffer in place around the canister by counteracting the swelling of the buffer*
- *to contribute to keeping the tunnels mechanically stable.*

The backfill should have chemically and mechanically favourable and predictable properties over long time periods and should not have any properties that could significantly impair the function of the other barriers. The desired properties include a sufficient density and low compressibility to keep the buffer in place, and a low hydraulic conductivity. The grain size distribution of the backfill should be such that significant intrusion of the buffer into the backfill does not take place. Some swelling *pressure (~100 kPa, corresponding to the weight of about four metres of loose rock) would be beneficial to support the tunnels against rock falls, to establish a tight backfill-rock contact and to counterbalance the effects of consolidation and piping in the backfill. Groundwater flow and transport of contaminants along tunnels and the excavation damaged zone (EDZ) around them could also be blocked by means of durable sealing structures. Buffering and sorption capacity for contaminants is a favourable property in the backfill material, too.*

The desired properties of the sealing structures include

- *no harmful effects on the other barriers*
- *chemical and mechanical stability over long time periods*
- *low hydraulic conductivity*
- *natural materials, like clay, with proven long-term properties are preferable to man-made materials.*

1.1 Workshop Presentations

The workshop included 10 presentations[1] in three sessions as follows:

1.1.1 Session 1 The Role of Backfill in Safety Analyses

1. **What are the requirements of the backfill from the point of long-term safety,** by Patrik Sellin, SKB
2. **Investigations of backfilling techniques and properties at Äspö HR*L;*** by Lennart Börgesson, Clay Technology AB
3. **Alternative backfilling concepts for a spent fuel repository at Olkiluoto** by Jorma Autio and Reijo Riekkola, Saanio & Riekkola Consulting Engineers; Jukka-Pekka Salo, Posiva Oy: Timo Vieno,VTT Energy.

[1] Dr. Paul Young (University of Liverpool), scheduled to present a paper ***Acoustic Emission measurements relating to triggering/stabilising microcracking,*** was unable to attend the workshop.

1.1.2 Session 2 Backfill operations and techniques for application

4. **What are the requirements of Sealing Performance** by Yutaka Sugita, JNC
5. **A new method for tunnel backfilling** by Tapani Kukkola, Fortum Nuclear Service Oy
6. **Techniques for applications of low permeable soils** by Heinz Thurner, Geodynamik AB

1.1.3 Session 3 Rock/Backfill interaction

7. **General aspects of groundwater flow in the EDZ** by Patrik Vidstrand, Chalmers University of Technology
8. **The role of backfill in suppressing brittle failure** by Derek Martin, University of Alberta
9. **Preliminary results from 3DEC modelling of a deposition tunnel** by Rune Glamheden, SYCON
10. **Confining stress and fracture growth** by Derek Martin, University of Alberta

The presentations were supplemented by discussion after each paper, and a general discussion at the end of the workshop.

1.2 Points from the Discussions

1.2.1 Papers 1-4

The role of backfill in safety assessment was a topic of particular concern in this session (Papers 1, 3 & 4) and throughout the Workshop. According to Sellin, (Paper No. 1) SKB Safety Analysis (SA) currently considers backfill to have a threefold purpose.

1. To limit expansion of the buffer material that surrounds each waste canister (emplaced vertically in the floor of the deposition tunnel).
2. To establish a low hydraulic conductivity along the entire tunnel system.
3. To generate a swelling pressure sufficient to ensure intimate contact between the periphery of the tunnel and the backfill, for the entire period (> 100000 years) of repository isolation.

It was also noted that SKB follows the ‘multiple-barrier’[2] (or defence-in-depth) principle used in many countries, and that backfill is not considered to be a separate barrier by SKB. The current goal (according to SA) should be to place the backfill such that the overall hydraulic conductivity along the (filled) tunnels is essentially the same as that of the rock mass between major conducting fractures. In other words, the backfilled tunnel can be assumed not to exist (i.e. the rock mass is ‘restored’) from the point of view of groundwater flow computations in Safety Analysis.

Posiva’s treatment of backfill in safety analysis (Paper 3) is generally similar to that of SKB, although backfilled tunnels are taken into account as potential pathways – allowing for consideration of various strategies to achieve acceptably low overall rates of flow. A combination of sections of high and low hydraulic conductivity sections of the tunnel, for example, may achieve an overall low flow rate more cost effectively than a strategy in which all sections are backfilled in the same manner. Since conductive fracture zones can be identified during construction it could be effective, for example, to give special attention to the sealing of these zones, thereby minimising access to regional pathways, rather than designing a uniform backfill procedure for all tunnel sections. The rock mass is not homogeneous, so why should not the backfill be modified correspondingly to achieve an optimum resistance to flow overall?
Special emphasis needs to be devoted to performance assessment of a repository, parts of which may, initially or in the long term, have a (significantly) higher hydraulic conductivity than the surrounding rock. (Paper 3, page 5)

The question of the ability of backfills containing some proportions of expansive clay to maintain a swelling pressure over the long term was raised. The possibility that saline groundwater could eventually replace fresh waters was given as an example of changes that may reduce the swelling potential. This could result in a compaction of the backfill and generation of a (highly conductive) gap in the upper part of the tunnel. Further, the general difficulty of placing backfill in horizontal tunnels such that there is intimate, and permanent- low hydraulic conductivity-contact with the roof presents a safety assessment concern.

It is recommended that the possibility of a gap between the tunnel periphery and the backfill be accepted, and that it be treated as part of the EDZ. Thus efforts should be made to minimise the gap, e.g. by use of expansive backfills (bentonite) above crushed rock or other techniques, and periodic seals be designed to interrupt the continuity of both the EDZ and a “presumed” tunnel/backfill gap.

Paper 4 discussed the approach taken to tunnel and shaft sealing in Repository Safety Assessment in Japan. Currently, as in Sweden, it is assumed that the backfill will be placed such that the overall hydraulic conductivity of the rock mass will be essentially unchanged from the intact (pre-mining) condition. The need for dialogue between performance (safety) assessment staff and repository design engineers- in order to arrive at an adequate performance of the overall system- was noted. This system may not necessarily optimise every sub-component of the isolation system.

[2] In which several separable barriers to radionuclide transport can be identified in the repository isolation system (e.g. canister, buffer material, rock mass,…)

1.2.2 Papers 2,5,6

This group of papers dealt with practical aspects of backfill emplacement.

Paper 2 noted that one of the aims of the Prototype Repository at the HRL, Aspo is to develop practical procedures for backfilling tunnels to achieve a hydraulic conductivity of the backfill $k < 10^{-9}$m/s, so that the tunnel does not become a preferential path for groundwater flow through the repository. Laboratory tests indicate that this level of conductivity (or lower) can be achieved with various bentonite/crushed rock mixtures at dry densities of 1.7 g/cm^3. [The deformation modulus of the backfill at this density was of the order of 50MPa. The modulus of elasticity of the rock mass is approximately 30GPa.] Densities achieved in the field tests to date are lower, and there have been difficulties achieving a uniform backfill density in the field. A dynamic compactor subjects the inclined face of the backfill pile to vibrational loads. Material discussed in this presentation was supplemented by a visit during the workshop to the site of the backfill tests and the plug/seal experiment in the Prototype Repository.

Paper 5 described some preliminary mini-scale tests to assess the possibility of forming a low conductivity backfill by injecting a viscous (setting) material into crushed rock preplaced in the tunnel. Some difficulties have been encountered in removing air from the voids to allow access of the viscous fluid. Civil engineers have developed vacuum injection techniques to ensure effective grouting in a number of practical applications. This may be studied further for this possible application.

Paper 6 discussed techniques used to compact soils in civil engineering (at surface sites). In later discussion, after the visit to the backfill test site, the author, Heinz Thurner cautioned that difficulties could be encountered in achieving homogeneous compaction if backfill was projected into the tunnel at high velocity (e.g. pneumatically or by high speed conveyor). Size segregation would occur with coarser particles falling to the bottom of the backfill pile.

1.2.3 Papers 7,8,9,10

This group of papers discussed a number of rock mechanics issues associated with backfilling of repository tunnels.

Paper 7 discussed observations of the EDZ in the ZEBEX experiment (1994-5) at the Aspo HRL, and possible hydrological interactions between an unsaturated EDZ and the backfill.

Results indicate that in the jointed rock mass of Aspo, the extent and hydraulic conductivity of the EDZ along the tunnel section was very heterogeneous (variable in extent and intensity).

This is consistent with the variability in size, attitude, joint properties of the rock blocks around the tunnel. Some blocks will tend to slip and become destressed while others will be heavily loaded-and likely to fracture to form an EDZ within the block. Note that analyses using discrete block models (UDEC; 3DEC) alone can not simulate such cracking within blocks. PFC discretisation of a heavily loaded block could produce internal fracturing such as may occur in EDZ development. It would be instructive also to examine the influence of thermally-induced stresses on fracturing induced around the canister hole and around the emplacement drift in the case of a discretely jointed rock mass. This could have implications for the design of plugs and seals. A continuum analysis of this problem has been carried out by Hakami and Olofsson (2000).

Paper 8 discussed stress concentration effects around emplacement drifts. The effect of in-situ stresses on greatly limiting the size of rock wedges likely to fall out of a roof fallout indicates that backfill pressures considerably lower than 100kPa should suffice to ensure stable openings in the tunnels. Intersections are less stable but 100kPa will still provide adequate support. Although TBM cut (circular) tunnels tend to be preferred over drill and blast excavations, a flat floor is more desirable for floor emplacement of waste canisters, as in the KBS3 concept.) A flat roof is also found to be more stable in deep mining practice.

Paper 9 discussed some preliminary results of an analysis to attempt to estimate the magnitude of non-elastic movements that could occur due to long-term degradation of the strength of the joints in a jointed granite rock mass. Since there is no information on how joint properies may deteriorate with time (extending to many thousands of years) the author assumed major reductions in the frictional and cohesive properies of the joints. The resulting displacements of the tunnel walls are almost an order of magnitude higher than usually observed in short term deformations. It would be interesting if the paper could include details of the radial extent of the inelastic deformation in the more severe cases. This analysis could provide an opportunity to attempt to obtain an estimate of joint degradation behaviour by analysis of ”Stand-up Time” data available for excavations in jointed granites.

The results indicating that a backfill pressure of the order of 100kPa did not inhibit the radial deformation of the tunnel was considered by some workshop participants as evidence that the backfill did not help to support and maintain the stability of the tunnel. This indicates a misunderstanding of the stabilising role of the backfill, which is to hold up detached rock blocks in the roof (or walls) and to inhibit fracture extension as discussed in Paper 10 (below).

The point to be made is that these (small) displacements are *elastic* values that occur *instantaneously* upon excavation of the unsupported tunnel. Equilibrium is established between the in-situ stresses, which are of the order of “tens of MPa” intensity, and the unsupported tunnel, only because of the high ***tangential*** (or arch) stresses that develop around the opening as the rock mass *wedges itself* tighter and tighter together with inward displacement. Even when the rock strength is reduced to very small values as in paper 9, the in-situ stresses remain unchanged at MPa levels and must be balanced by the development of comparable *MPa-level* tangential stresses in the rock around the excavation. Weakening of the rock, as considered in paper 9, simply increases the displacements needed to develop the required tangential stress levels. Addition of a backfill support pressure of the order of 0.1 MPa can have negligible effect on restricting the inward movement of the rock wall as it degrades in strength over time. As noted in paper 3 (p.4 lines 16,17), some swelling pressure (~100kPa, corresponding to the weight of about four metres [height] of loose rock would be beneficial to support the tunnels against rock falls, to establish a tight backfill.

During the Workshop, Dr Pusch indicated that a backfill pressure of 100kPa was established to support the maximum credible height of rock that could fall out of a tunnel roof at depth in a fractured granite, such as may be encountered in Scandinavia.

Paper 10 presented observations on the effects of small changes (20kPA ~100kPa) in confining pressure on crack growth (as indicated by micro-seismic activity emanating from the cracks) in brittle rock (e.g. granite at the Aspo HRL). Brittle fractures, such as those in the EDZ, are known to be in metastable equilibrium. Very small changes e.g. in humidity, chemical composition of tunnel water or atmosphere, and/or confining pressure can arrest or trigger crack extension processes. Micro-seismic techniques (such as those developed by Paul Young) are now able to detect such micro-cracking processes and have demonstrated that small changes in confining pressure (<100kPa) are quite adequate to inhibit the extension of cracks in the EDZ-and prevent increase in the hydraulic conductivity of the zone. However, such small pressures will probably have little influence on *closing* existing fractures (and hence will not reduce the hydraulic conductivity of the EDZ). Paper 9 also discussed briefly the use of paste backfills in mining. These may have application in repository backfilling. A copy of the paper by Pierce et al. (1998) was distributed to all participants.

1.2.4 Main conclusions

1. The treatment of backfilled tunnels in Safety Assessment should be carefully re-examined, including the relative theoretical and practical merits of continuous backfill, versus backfill with plugs and seals. Intelligent placement of plugs e.g. at sections where a main conductive fracture intersects the tunnel, could lead to more effective protection against radionuclide transport than a uniform backfilling of the tunnel.

2. Although a uniform degree of compaction may be difficult to achieve practically in backfilling of horizontal tunnels, local variations in compaction should not compromise the overall provided that they are local i.e. non-continuous, and not too large.

3. A realistic upper bound of compaction and associated hydraulic conductivity needs to be established.

4. The consequences of heterogeneous compaction on the overall hydraulic conductivity of continuously placed backfill need to be analysed.

5. The influence of groundwater salinity on the performance of expansive clays and clay/crushed rock combinations for tunnel seals needs to be examined further.

6. Long-term degeneration of joint properties may contribute to the maintenance of a tight contact between backfill (or plugs/seals) and the rock. However, the initial estimates presented at the workshop (Paper 9) greatly over-estimate both the magnitude and uniformity of long-term closure.

7. Micro-seismic data suggests that relatively small (~ 100 kPa) backfill pressures are sufficient to inhibit extension of fractures in the EDZ during the post-thermal cycle (> 2000 years or so) period of isolation. Further study is needed to confirm the general validity of this potentially important observation.

8. Thermally induced stresses in the rock around the tunnel periphery may extend the EDZ somewhat. This should be considered in the design of plugs and seals.

9. Long-term creep caused by gradually changes of the strength properties of joints may cause loads than could be not supported by the backfill. In such a case will the backfill be compacted as the joints may open. More knowledge of the possible mechanisms for gradually change of the strength properties of joints by alteration may be needed.

10. Examination of backfill technologies and recent innovations in the mining industry may provide valuable insights and economic alternative backfilling strategies.

11. Three dimensional discrete fracture modelling of the tunnel excavation region (in combination with PFC modelling of some blocks) that intersect the tunnel could provide insight into the heterogeneous nature of the EDZ and its lateral continuity as a connective hydraulic pathway.

2 Backfilling with mixtures of bentonite/ballast materials or natural smectitic clay?

ATTACHMENT TEXT EXTRACTED FROM:

(TR 98-16, Roland Pusch, Geodevelopment AB, provided by the author after the workshop)

2.1 Summary

The report presents the outcome of a comparison of the performance of backfills of mixed MX-80 and crushed rock ballast, and a natural smectitic clay, represented by the German Friedland clay.

Microstructural models for mixed backfills implying that the MX-80 grains are preserved do not work since the grains break down. The clay component of bentonite/ballast mixtures fills the large majority of the space between the ballast grains even at 10 % MX-80 content but the homogeneity and density vary strongly. Mixtures with 30 % MX-80 compacted to an achievable density is just about sufficient to provide the required rock support and resistance to upward expansion of buffer in the deposition holes and to make the backfill tighter than the surrounding rock. In all these respects the investigated natural clay is superior.

Grains of crushed TBM muck as ballast have very fine rock debris attached to them which forms more or less continuous coatings that provide flow paths and hence raises the bulk hydraulic conductivity.

Both technically and economically it appears that the Friedland clay is a competitive alternative to mixtures of 30 % MX-80 and crushed ballast. However, it remains to be demonstrated on a full scale that Friedland clay ground to a suitable grain size distribution can be acceptably compacted on site.

2.1.1 Quantification of requirements

1. The backfill must be dense and have a low compressibility. Upward expansion of the buffer takes place until equilibrium is established between the swelling pressure of the buffer and the effective contact pressure mobilised in the overlying backfill. A preliminary measure of the maximum allowable displacement of the interface between the buffer and backfill is 20 cm.

2. The swelling pressure of the backfill should be 100 kPa for preventing rock fall (blocks with 3 m height) from the roof and for establishing a tight backfill/rock contact.

3. The criterion that the backfill should not be an important hydraulic conductor, means that it should be less permeable than the surrounding rock. This condition may not be applied in due time since it is still not known whether a combination of permeable backfill in the form of crushed rock and strategically located, very tight plugs provide

the same limited groundwater flow as very tight clayey backfills. The plug-based concept is not considered here.

Additional parameters of less importance are practicality in handling, transport and application, and cost.

2.1.2 Application

The backfill material must be effectively compactable by use of ordinary techniques available to contractors. The presently investigated techniques using blades for moving the backfill material to form inclined layers in the tunnel and vibrating plates for lateral compaction should be applicable.

2.2 Candidate materials

2.2.1 Principle of selection

Backfill of KBS3 type, i.e. mixtures of MX-80 bentonite and non-clay ballast, is a given candidate. A competing candidate material group is represented by natural smectitic clay and the Friedland clay from the Neubrandenburg area in Mecklenburg-Vorpommern, Germany, is taken as a representative of this type of material. Major reasons for this choice are the large amount of accessible material and extensive use of Friedland clay for environmental protection and also the fact that the clay is very well characterised and has been used as raw material in advanced tile manufacturing on an industrial scale for decades.

2.2.2 Major differences between mixed backfills and Friedland clay

It is concluded from the laboratory studies that the evaluated hydraulic conductivity of mixtures of MX-80 and ballast varies strongly for any density, while data from the fewer Friedland clay tests indicate less variation. This is clearly related to incomplete filling of clay of the voids between the ballast grains in the mixtures and to significant variations in density of the clay component. In mixtures of MX-80 and crushed rock ballast fine debris prevents the clay from filling the void space and adhering to the solid ballast grains, which is believed to raise the bulk conductivity. At incomplete clay filling the risk of piping and erosion is also increased and it is therefore essential to obtain a high degree of clay filling of the voids between ballast grains, which is naturally more difficult for low bentonite contents.

No criterion has been set as to the required hydraulic conductivity of the backfill but taking the average hydraulic conductivity of the rock around the tunnels to be on the order of 10^{-11} m/s (excluding the excavation-disturbed zone) as in Stripa granite, backfills of 30 % MX-80 and crushed rock ballast mixtures with the achievable density 2070 kg/m^3 at saturation will not do, while Friedland clay fulfils the requirement. If the rock conductivity is 10^{-10} m/s, 30/70 mixtures saturated with strongly brackish water dominated by Ca will provide the same bulk conductivity as the rock, while Friedland clay backfill with a density of 2000 kg/m^3 at saturation will have a bulk conductivity that is 10 times lower than that of the rock. If the rock conductivity is 10^{-9} m/s, mixtures with 20 % MX-80 will do, while mixtures with 10 % MX-80 can not be accepted unless the rock conductivity is as high as about 10^{-7} m/s.

2.2.3 Chemical performance

The chemical data specified in sections 3.2.3 and 3.3.3 imply that the contents of SiO_2 and Al_2O_3 in the Friedland clay are about 10-15 % lower than that of MX-80 while the concentration of other elements are very similar. The clay components are hence comparable. Including also the ballast material in the mixed backfills the silica and aluminum concentrations will increase but also the amount of potassium, calcium and iron will be raised to higher values than those valid for Friedland clay.

As to the mineral content the differences in clay mineral content is the most obvious feature. Considering montmorillonite in the first place, Friedland clay has a slightly smaller amount than backfills with 30 % MX-80, which, theoretically, make the microstructure of the latter type of backfills more homogeneous. However, the fact that Friedland clay also contains expandable minerals of other kinds (mixed layer minerals) the mineralogy of this clay is superior as manifested by the microstructural homogeneity as well as by the higher swelling pressure and lower hydraulic conductivity at the respective reference densities.

2.2.4 Cost

The total required mass for backfilling of tunnels and shafts in a KBS3 repository is on the order of 1000 000 tons. The approximate cost of MX-80 bentonite is 2500 SEK per ton per August 1998, which means that even backfills with only 10 percent bentonite cost well over 250 MSEK to which comes expenses for the major backfill component, handling and application. The total material cost for fully prepared backfill with 10 % bentonite will be at least 300 MSEK, while it rises to 800-1000 MSEK for mixtures with 30 % bentonite. Using cheaper commercial montmorillonite-rich bentonites the cost for 30 % clay mixtures may be brought down to 600-800 MSEK.

The alternative candidate material type, natural smectitic clay, represented by the Friedland clay in the present study, is clearly competitive. Thus, referring to the price offer given by the German manufacturer DURTEC in early 1998, the material cost will be 700 SEK per ton, i.e. about 700 MSEK for 1000 000 tons.

2.3 Microstructural aspects

2.3.1 Theoretical microstructural modelling using FEM

Finite element technique (FEM) has been applied for modelling the microstructural evolution of bentonite/ballast backfills. It was based on the assumption that the bentonite grains are equal and spherical with 1 mm diameter and that the ballast grains are also spherical but with 2.4 mm diameter. The weight ratio bentonite to ballast was 1 to 10. In 2D the structural arrangement was as shown in Figure 12.

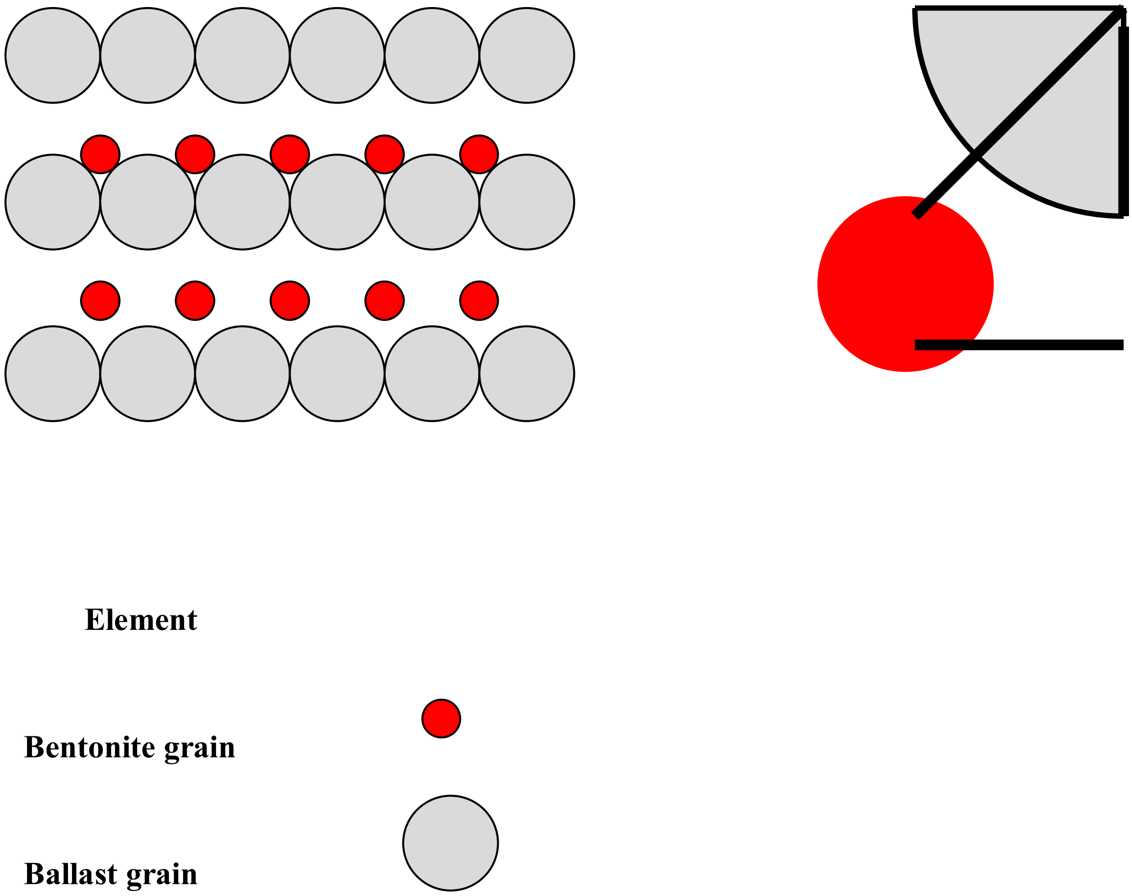

Figure 1. *Bentonite/ballast mixture with 10 % bentonite.*

Calculations using ABAQUS and a material model assuming modified Drucker-Prager plasticity, ”porous elasticity”, porewater pressure related to density-dependent suction (swelling pressure with negative sign) and hydraulic conductivity related to the void ratio show that the bentonite grains will expand to fill 75 % of the initially water-filled void in the element in 20 seconds. After infinite time the unfilled parts, which are located at the contacts of ballast grains, represent about 0.5 % of the initial void. These isolated clay-filled voids do not contribute much to the bulk conductivity but the outer parts of the bentonite grains are soft (e=1.6-3.5) and permeable and cause a rather high conductivity ($K>10^{-9}$ m/s). The expandability and swelling pressure of 10/90 backfills are almost nil even at 2100 kg/m^3 at saturation.

Increasing the bentonite content to 15 % and using fine-grained bentonite (0.5 mm) the microstructural model in Figure 13 is obtained. The ballast grains are still in contact but the voids have been reduced from about 1 mm maximum diameter to about 0.5 mm. The density of the clay filling in the ballast voids is somewhat higher but the net bulk hydraulic conductivity is not significantly reduced ($K>5^{-10}$ m/s). The expandability and swelling pressure of 15/85 backfills are insignificant. The latter is in the range of 50-200 kPa for a density at saturation of 2100 kg/m^3.

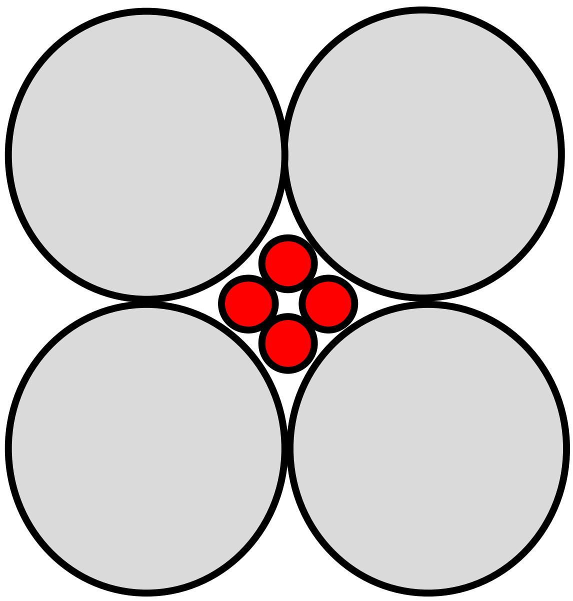

Figure 2. *Microstructural model in 2D of MX-80/ballast mixture with 15 % fine bentonite grains.*

Not until the bentonite content is raised to about 30 % the hydraulic conductivity K is significantly reduced and the swelling pressure p_s appreciably raised: K>e-11 m/s and $100<p_s<400$ kPa for a density at saturation of 2000 kg/m^3. The ballast grains are then not in contact anymore but float in the clay matrix. The homogeneity and density are then the sole conductivity-controlling parameters except for the porewater chemistry.

2.4 Improved 3D modelling of mixtures of MX-80 bentonite and ballast material

2.4.1 Mixing

The relative amounts of bentonite and ballast particles smaller than 1 mm in fact determine the microstructural constitution. Mixtures of MX-80 grains and crushed rock ballast can be approximated to consist of two grain sizes, 400 μm and 100 μm, the largest representing the ballast and the smallest the bentonite. A 10/90 bentonite/ballast mixture contains about 8 times as many bentonite grains than ballast grains, while for a 30/70 mixture this ratio is 24. On compaction the clay grains are moved into the voids between the ballast grains and also form coatings (cutans) of these grains if the water content of the mixture exceeds a few percent.

Preparation and compaction of mixtures, air-dry or wetted, imply strong agitation by which bentonite grains undergo abrasion and break into fragments. The size of the bentonite grains after such agitation ranges from a few micrometers to a tenth of a millimetre in the finer part of the mixtures. The rotation and sliding of clay and ballast particles caused by vibratory movement and a large number of cyclic pressurising bring them into dense layering. These processes invalidate FEM calculations based on the assumption that the initial bentonite grains are unaffected. Application of BEM with due consideration to 3D conditions has therefore been made as described in the report.

2.4.2 Influence of bentonite content

In principle, an element of backfill is as indicated in the figure below:

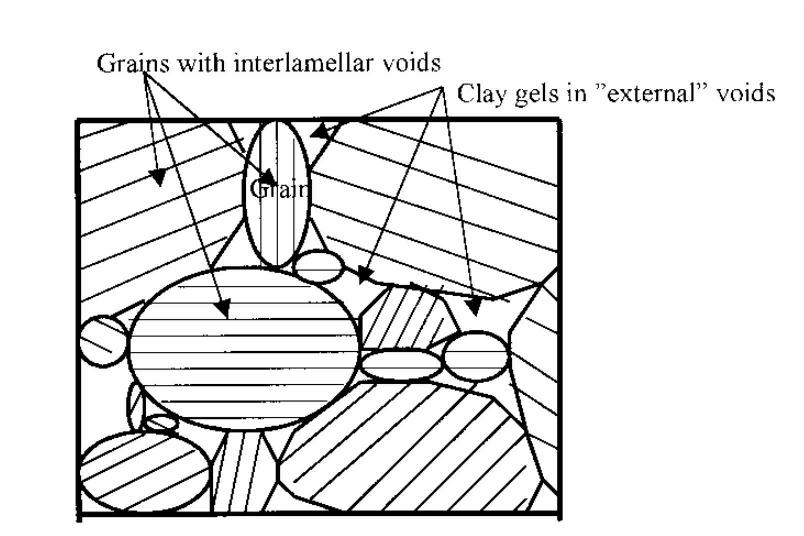

Figure 3. Generalised microstructure of soil for backfilling purposes.

If it consists of mixtures of MX-80 clay powder grains and ballast grains the majority of the bigger grains are non-expandable but if it is a smectitic clay like Friedland Ton, all grains expand on hydration and fill the initial voids with clay gels that have different densities, depending on the clay content and the void size, which is a function of the bulk density. For the latter type of clay the density of the clay fraction does not deviate much from the bulk density, while for MX-80 mixtures the density of the clay fraction is much lower than the bulk density when the clay content is lower than about 30 % by weight. This difference is the major reason for the difference in hydraulic conductivity and swelling pressure.

For backfill with 30 % bentonite the clay forms a matrix that controls the bulk hydraulic conductivity and swelling pressure. The microstructural evolution is largely controlled by the interaction of bentonite grains and a unit cell of the backfill. For modelling it is required to take the grain size variation of the bentonite into consideration, which explains that two grain diameters appear in the calculations, i.e. 350 and 100 µm. The bigger grain is taken to be 400 µm, which makes the geometries of the 10/90 and 30/70 backfills identical.

2.4.3 Compaction

Assuming that compaction with plate vibrators yields a dry density of 1750 kg/m^3 (2100 kg/m^3 at water saturation), the volume of the unit cell of the microstructural model is decreased by 15 %. Like the 10/90 backfill uniaxial compaction gives a shortening of the unit cell by 30 µm and a reduction of the volume of the void in the unit cell to $2.6x10^6$ μm^3 in conjunction with relative movement of the grains and change of the cubical symmetry.

Unlike the 10/90 backfill the big grain is deformed plastically, which makes the small grain deform less and remain coherent although strongly deformed. The main difference between the two cases is that both grains expand and produce a clay gel with higher density than in the 10/90 case. The stress/strain behaviour of the grain system has been modelled by use of the BEM code BEASY.

The density of the gel, which will consist of nearly fully expanded stacks of lamellae with $3x10^{-6}$ μm^3 free water between them, will have a density of 1890 kg/m^3. It has a hydraulic conductivity of about $2x10^{-13}$ m/s at percolation with distilled water and 10^{-12} m/s at percolation with 3.5 % $CaCl_2$ solution, yielding a bulk conductivity of $6x10^{-14}$ m/s at percolation with distilled water and $3x10^{-12}$ m/s at percolation with 3.5 % $CaCl_2$ solution. These values are also lower than experimental data by about 2 orders of magnitude, which demonstrates that the clay gel is not homogeneous.

2.4.4 Modelling of Friedland clay

The grain size of the investigated finely ground air-dry Friedland clay is significantly smaller than that of the clay component of 30/70 MX-80/ballast and the ratio between the numbers of small and big clay grains that make up a representative generalised microstructural unit cell is higher. This means that the voids between the big grains are more effectively filled with small grains and the remaining voids are hence smaller for any bulk density value. For the density at saturation of 2000 kg/m^3 that is expected to be achieved in tunnels, the compaction of a unit cell of Friedland clay would cause uniaxial shortening by about 20 %.

In contrast to the MX-80 grains, those in Friedland clay contain less expandable minerals (about 50 %) that may not be equally represented in all grains. This would imply that the clay gel formed by coagulation of dispersed grain fragments is more heterogeneous and permeable than in backfills of 30/70 mixtures. However, since the space between the grains is smaller than in such mixtures matured Friedland clay is still assumed to be at least as homogeneous as 30/70 backfills.

Appendix

Backfill program

A POSIVA/SKB Workshop on

BACKFILL REQUIREMENTS IN A KBS-TYPE REPOSITORY*

AUGUST 27-28 2001
ÄSPÖ Hard Rock Laboratory, Sweden

Welcome

The focus of the workshop is to: benchmark our progress in repository design; highlight issues related to backfill/rock interaction; and provide a forum for discussion of the issues of importance in safety assessment.

Key questions to answer are:

1. What are the requirements of the backfilling from long-term safety assessment?
2. How can these requirements be satisfied?
3. What is known of the backfilling?
4. What more do we need to know?

Additional to each Talk, should a paper be submitted. These papers will be printed in a joint report and presented at the ONDRAS/NIRAS 6th International Workshop on Design and Construction of Final Repositories "Backfilling in Radioactive Waste Disposal" in Brussels, 11 – 13 March 2002 by Rolf Christiansson, SKB.

Schedule

Monday Aug 27		**Tuesday Aug 28**	
09.00	Transport from Hotel Post, Oskarshamn to Aspo HRL	07.45	Transport from Hotel Post, Oskarshamn to Aspo HRL
09.30	**Registration & Coffee**		
10.00	**Introduction**	**08.30**	**Session 3**
10.15	**Session 1**		
		10.00	**Coffee**
12.00	**Lunch**		
		12.30	**Lunch**
13.00	**Field excursion**		
		13.30	**Chaired discussion**
15.00	**Coffee**		
15.30	**Session 2**		**Coffee**
≈ 17.30	End of day- Transport to Hotel Post, Oskarshamn	**16.00**	**Closure of Workshop**
20.00	**Dinner at Hotel Corallen**		

Talk programme

Each presentation should account for 25 minutes and will be followed by a short time for questioning. Each session will be closed with a summary.

Session 1 The Role of Backfill in Safety Analyses

Talk 1 "What are the requirements of the backfill from the point of long-term safety"
Patrik Sellin, SKB

Talk 2 "Alternative backfilling concepts for a spent fuel repository at Olkiluoto"
Jorma Autio, Reijo Riekkula, Jukka-Pekka Salo & Timo Vieno

Field excursion.

"The KBS3 backfilling concepts – field excursion and additional presentations"
Personnel from Clay Technology

Session 2 Backfill operations and techniques for application

Talk 3. "What are the requirements of Sealing Performance"
Yutaka Sugita

Talk 4. "A new method for tunnel backfilling"
Tapani Kukkola

Talk 5. "Techniques for applications of low permeable soils"
Heinz Thurner

Session 3 Rock/Backfill interaction

Talk 6. "General aspects of groundwater flow in the EDZ"
Patrik Vidstrand

Talk 7. "The role of backfill in suppressing brittle failure"
Derek Martin

Talk 8. "Preliminary results from 3DEC modelling of a deposition tunnel"
Rune Glamheden

Talk 9. "Confining stress and fracture growth"
Namkak Cho

Talk 10. "Acoustic Emission measurements relating to triggering/stabilising microcracking"
Paul Young

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Paper 1

What are the requirements of the backfill from the point of long-term safety?

Patrik Sellin, Swedish Nuclear Fuel and Waste Management Company (SKB)
Stockholm, Sweden

Abstract

Although not a barrier itself, the tunnel backfill is and important part of the KBS-3 system. The purpose of the backfill is to ensure that the buffer and rock will retain their favourable properties. This paper describes the requirements on the backfill from the point of long-term safety, the development of the backfill material and discusses future requirements and solutions.

Current requirements

The backfill is still to ensure that the buffer and the rock will maintain their desired properties. The primary requirements are [1]:

- The backfill should have a density which minimises the upward expansion of the buffer
- The hydraulic conductivity should be comparable to that of the surrounding bedrock
- The swelling pressure should be at least 100 kPa at the tunnel roof
- Long-term stability and no negative effects on other barriers

The backfill in the deposition tunnels is no barrier itself in the KBS-3 concept. The ideal situation (figure 1) would be to have a repository without deposition tunnels and access tunnels/shafts, but there is currently no known engineering solution for that. The present KBS-3 layout can be found in figure 2.

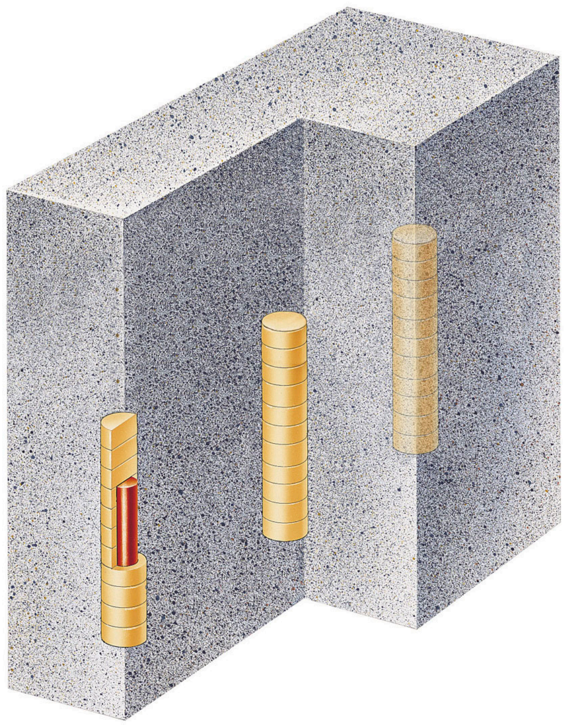

Figure 1. *Ideal KBS-3 Near Field*

The tunnels are filled with a mixture of crushed rock and 10-30% bentonite. Ideally, the tunnels should be filled with a material, which restores the situation to what it was before the excavation.

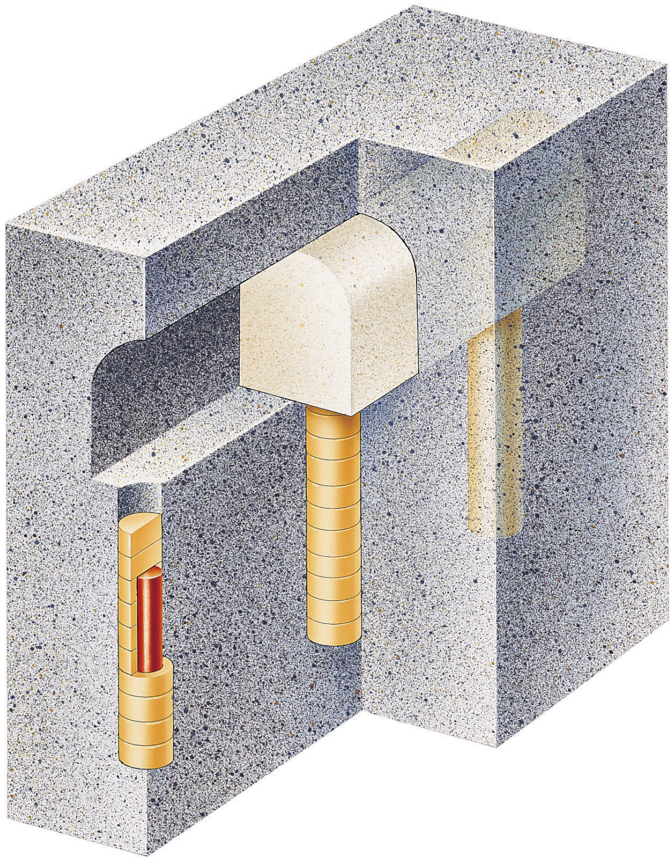

Figure 2. *Current KBS-3 Near Field*

The backfill should limit the upward expansion of the buffer. In the interface between the buffer and the backfill, the buffer exerts a swelling pressure against the backfill and *vice versa*[2]. Since the difference in swelling pressure is great, a net pressure arises against the backfill whereby the buffer swells and the backfill is compressed. In this process, the swelling pressure from the buffer decreases as the density decreases. At the same time, the counter-pressure from the backfill increases as it is compressed and its density increases. The swelling of the buffer and compression of the backfill are counteracted to some extent by friction against the rock. When the force of the swelling pressure in the buffer is equal to the sum of the force of the counter-pressure in the backfill and the friction against the rock, the process ceases since equilibrium has been established. The size of the swelling depends on the original densities of the buffer and the backfill and associated expansion and compression properties.

The hydraulic conductivity should be about equal to that of the surrounding rock on a repository scale. The transport resistance in the rock is an important barrier for radionuclide transport in the KBS-3 concept and the backfill should not be an important transport path for radionuclides. A number of uncertainties exist for bentonite-containing backfill [2]. The properties of the backfill are measured directly after mixing. The risk and effect of possible homogenisation of the bentonite density in the aggregate pores is not known. The measured hydraulic conductivities assume that there are no channels or gaps.

The swelling pressure and weight of the backfill act as a support against rock fallout but otherwise exert too little pressure to affect the rock [2]. The pressure is greatest in the floor, if the same material is used in the whole tunnel, since compaction is expected to be best and the weight of the backfill will be greatest near the floor. The swelling pressure is dependent partly on what backfill composition is chosen and partly on the salinity of the groundwater. An aim is to achieve at least 100 kPa of swelling pressure against the roof in order to resist some block fallout and to obtain a residual swelling capacity which can seal the possible effects of piping and creep movements in the backfill.

History

SKB has done a number of safety assessments over the years. This is a brief summary on how the backfill has been treated.

The KBS-3 concept has its origin in KBS-1 [3]. The KBS-1 concept consisted of three barriers: the vitrified waste form, the titanium canister and the rock. The canisters were surround by a buffer made of 20% bentonite and 80% quartz sand, except in the bottom below the canister where the quartz content was higher. The tunnels were filled with the same material. The buffer was not considered a barrier in KBS-1. Its main purpose was to centralise the canisters in the deposition holes.

Spent fuel as a waste form was introduced in KBS-2 [4]. The increased radionuclide inventory required additional barriers and therefore a buffer of highly compacted bentonite was introduced. However, the backfill was kept the same as in KBS-1 and was not discussed further.

In KBS-3 [5] requirements on mechanical support for the rock and hydraulic conductivity was introduced. It was stated that the backfill should have at least as low conductivity as the rock it replaces and a value of 10^{-9} m/s was given. No value was given for the swelling pressure.

In SKB 91[6], the essential properties for the backfill material was defined as:

- Low hydraulic conductivity
- Low compressibility
- Long term stability, so that the material retains its properties without having a detrimental effect on the environment in the deposition holes

No values were given for the properties. The backfill material from KBS-1 remained unchanged.

In SR 97[7] the quartz sand in the backfill was replaced by crushed rock. The composition was 15% bentonite and 85% rock. The long-term safety criteria for the backfill said that it should have:

- a hydraulic conductivity that does not significantly exceed the average conductivity of the surrounding rock, i.e. it should be around 10^{-10} m/s for typical conditions in Swedish bedrock.
- a swelling pressure of at least 0.1 MPa against the tunnel roof to support the rock around the tunnels[2].

The backfill should also prevent the buffer from protruding up into the tunnel when it swells. There were absolute criteria for the properties of the backfill in this respect.

It is interesting to note the backfill has remained virtually unchanged for 25 years. The long-term evolution of the backfill is not treated in any of the assessments. In SR 97 it is stated that this needs to be further examined in future safety assessments.

Importance of backfill

The importance of a high conductivity in the backfill for the radionuclide transport in the backfill has been evaluated [8]. It was found that the water flow rate in the tunnel has little influence on the release of radionuclide from the near-field, since the main transport resistance is not located in the tunnel. In general, the transport resistance in the small damage on the canister wall controls the release from the canister into the geosphere before the canister collapses. When the canister has collapsed, the transport resistance is mainly found in the bentonite around the canister in the deposition hole.

The near field release increases by a factor of two for some radionuclides when the water flow increases from 0.01 to 1.0 m^3/year. In order to test the impact of a high water flow rate, calculations were performed for a water flow rate of 10.0 m^3/year. Results show that the release increases only slightly (See Figure 3).

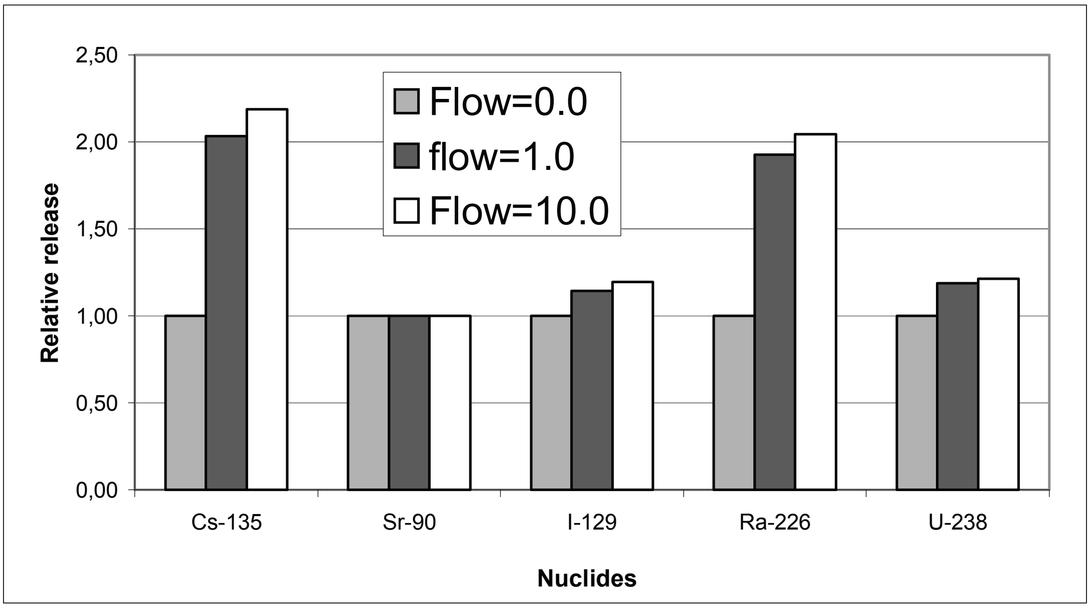

Figure 3 *Relative release rate as a function of the water flow rate in the tunnel. A high flow rate is included for the sake of comparison.*

In this context, it is important to note that the impact of a high flow in the backfill on the geoshpere performance has not been studied.

Future requirements

The backfill was found to be one of the weakest points of the KBS-3 concept in SR 97 [7]. The requirements on conductivity and rock support are essential to ensure that that the natural fractures in the rock will be the most important transport path from the repository, even for very long time scales. An early backfill degradation would be a violation of the multiple barrier principle, since that could lead to a loss of buffer/rock barrier.

The long-term properties of the backfill have to be studied in more detail in the future. Some processes that might be of importance are:

- Ion-exchange from sodium to calcium. This process is of minor importance in the buffer, but might be important in the backfill, since the bentonite content is much lower

- Salinity. A high salinity in the groundwater may remove all the swelling capacity from the backfill.

- Erosion. Is not considered to be a major problem, but could occur if channels are formed in the backfill, or in a situation with an extremely low ionic strength groundwater (glacial melt water)

These processes could potentially lead to a loss of the required properties of the backfill. The backfill is of importance in all time scales of repository evolution. Therefore, the stated requirements should be met even after a time period of 100 000 years or more. An initial swelling pressure of 100 kPa and a hydraulic conductivity of $\sim 10^{-10}$ m/s may not be sufficient when the time dependent evolution is taken into account.

There are a couple of alternatives that could improve the long-term performance of the backfill. Some examples are:

- Siting of the repository at an inland site with a low ionic strength groundwater. This will limit the effects of salinity on swelling pressure and also create a more favourable situation for ion-exchange.

- A higher bentonite content in the backfill. This would increase the swelling pressure and decrease the conductivity. The negative effects are an increased cost and an increased compressibility.

- Backfilling with a natural clay material. This could be a good alternative from a cost and performance point of view, but has not been fully investigated yet.

- Horizontal deposition. Limits the use and importance of backfill material.

The long-term performance of the backfill has not been given very much attention, since the KBS-3 concept was invented. The current solution is not satisfactory from a safety point of view. The questions about backfill requirements, design and long-term performance should be solved well in time before the next full safety assessment, due in 2007.

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Paper 2

Alternative backfilling concepts for a spent fuel repository at Olkiluoto

Jorma Autio & Reijo Riekkola, Consulting Engineers Saanio & Riekkola
Jukka-Pekka Salo, Posiva Oy, Timo Vieno, VTT Energy

Overview on Posiva's plans

Posiva plans to construct a KBS-3 type repository for spent fuel in the crystalline bedrock at the island of Olkiluoto at the coast of the Baltic Sea. The Olkiluoto nuclear power plant and the VLJ Repository for low and intermediate level waste are also located on the island. Since 1987 site investigations for spent fuel disposal have been carried out at Olkiluoto, and in the different phases of the site investigation programme at five other sites. The Finnish Parliament ratified in May 2001 the Government's positive Decision in Principle on Posiva's application to locate the repository for spent fuel at Olkiluoto. The repository may accommodate 2600 – 4000 tU spent fuel from the Loviisa (2 x 440 MWe VVER-440 type PWR) and Olkiluoto (2 x 880 MWe BWR) nuclear power plants. Disposal of additional spent fuel from any new nuclear power reactors, currently under consideration in another Decision in Principle process, is part of the subsequent decision-making.

Construction of the Underground Rock Characterisation Facility ONKALO for detailed characterisation of suitable rock volumes for the repository is planned to be started in 2003 – 2004. The access routes into ONKALO and other underground excavations may later be used as parts of the repository. Access to the underground facilities may be arranged via a ramp and shafts or via shafts only. Posiva aims to submit the construction permit application for the repository around 2010, and later the application for the operation license, so that the operation of the repository could be started around 2020.

Figure 1 shows an illustrative example of a KBS-3 type repository layout at the depth of 500 metres at Olkiluoto. Preliminary layouts adapted to the present version of the bedrock model have been sketched for several alternatives, including different disposal depths, two-layer layouts, and the Medium Long Hole (MLH) concept where canisters are emplaced in horizontal deposition holes with a length of about 200 metres (Äikäs & Riekkola 2000).

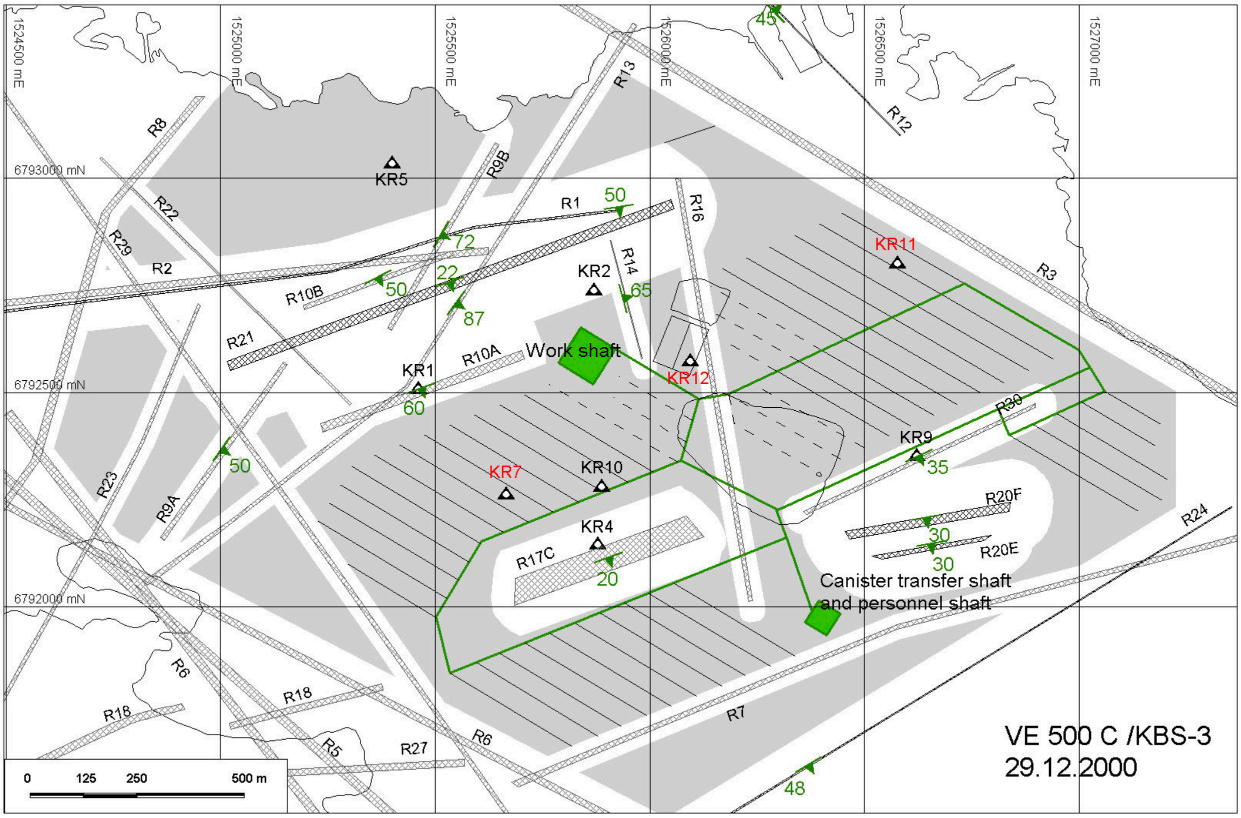

Figure 1. An illustrative example of a KBS-3 type repository layout at the depth of 500 metres at Olkiluoto (Äikäs & Riekkola 2000).

Geochemical and rock mechanical conditions at Olkiluoto

After the most recent glaciation, the island of Olkiluoto rose from the Baltic Sea 2500 to 3000 years ago. The layered sequence of groundwaters can be related to climatic and shoreline changes from modern time through former Baltic stages to the deglaciation phase about 10 000 years ago and even to preglacial times. Fresh groundwater (content of Total Dissolved Solids (TDS) < 1 g/l) is found to the depth of about 150 metres, and brackish (TDS between 1 and 10 g/l), sulphate-rich groundwater between 100 and 400 metres. Deeper groundwaters are saline (TDS > 10 g/l). At the depth of 500 metres, TDS varies between 10 and 25 g/l. The most saline waters at depths greater than 800 metres have TDS values between 30 and 75 g/l. These deep saline waters seem to have been undisturbed during the most recent glaciation and even much longer in the past.

Conditions in the deep bedrock are anoxic and reducing. With increasing depth the geochemical conditions shift from the oxic conditions near ground surface to post-oxic in the upper part of the brackish zone, and further to sulphidic in lower part of the brackish zone and in the upper part of the saline zone. In the deep saline groundwater, methanic geochemical conditions prevail.

The topography of the Olkiluoto area is rather flat. The highest point on the island reaches 18 metres above sea level, and the maximum height of the groundwater table approx. 10 metres above sea level. The rate of the land uplift is currently approx. 0.6 mm/yr. Fresh water infiltrating at the surface gradually displaces brackish and saline groundwater in the bedrock. Olkiluoto is likely to become an inland site with brackish or saline groundwater at the depth of 500 metres within the next 10 000 years (Löfman 1999, 2000).

During the construction and operation phases groundwater will be drawn into ONKALO and later into the repository from the surrounding bedrock. As a consequence, more saline groundwaters, presently laying 100 to 200 metres below the repository level, may rise to the disposal level (Svensson 1997). After the closing of the repository the salinity distribution will gradually return towards the natural state. During the glacial cycle groundwater salinity may increase, for example, during freezing of groundwater into permafrost, when dissolved solids concentrate in the remaining water phase (Gascoyne 2000), or in a situation where deep saline groundwaters from under the centre of the glacier are pushed to the upper parts of the bedrock at the periphery of the glacier (Svensson 1999).

A design basis TDS value of 35 g/l has been recommended for a repository excavated at the depth of about 500 metres at Olkiluoto (Vieno 2000). All repository systems and engineered barriers should perform properly at least at groundwater salinities ranging from fresh water to 35 g/l. Today the salinity at the depth of 500 metres varies from 10 to 25 g/l. A design basis value of 35 g/l would allow intrusion of groundwaters presently lying 100 to 200 metres below the 500-metre level. As 35 g/l is the salinity of ocean water, it would also take into account the maximum possible salinity of water infiltrating at the surface at any phase of future glacial cycles. If the repository were planned to be constructed deeper in the bedrock, the design basis salinity value needs to be raised. For example, if the repository would be located at a depth of 700 metres, a possibility of intrusion of highly saline, brine-type groundwaters (TDS nearing or exceeding 100 g/l) into the repository should be taken into consideration (Vieno 2000).

At the depth of about 500 metres, the majority of the rock mass at Olkiluoto consists of intact rock between fracture zones and has been estimated as “normal” from the point of constructability (Anttila et al. 1999). This means that excavation and rock support can be carried out using conventional methods and materials generally used in underground constructions. Close to the depth of 700 metres, the rock has been estimated as “demanding” or “very demanding” in terms of its constructability, because of its relatively low uniaxial compressive strength (σ_{ucs} = 80 – 140 MPa) and the higher stress field (σ_H = 35 – 40 MPa at 700 metres). Severe spalling of the rock may occur, and special methods and materials (including, for example, shorter blasting rounds, temporal supporting with bolts and shotcrete, and pregrouting) may be needed when excavating and supporting the rock.

The repository is planned to be constructed at the depth between 400 and 700 metres. Salinity of deep groundwater and rock mechanical constructability (stress/strength-ratio) favour the upper part of this range.

Functions and desired properties of backfill and sealings

The backfilling and sealing arrangements aim to re-establishment of undisturbed conditions in the geosphere (Posiva 2000). The main function is to prevent the disposal tunnels and access routes into the repository for becoming major conductors of groundwater and transport pathways of contaminants, and to block inadvertent intrusion into the repository. In a KBS-3 repository the functions of the backfill in the upper part of the deposition hole and in the tunnel further include

- to keep the buffer in place around the canister by counteracting the swelling of the buffer
- to contribute to keeping the tunnels mechanically stable.

The backfill should have chemically and mechanically favourable and predictable properties over long time periods and should not have any properties that could significantly impair the function of the other barriers. The desired properties include a sufficient density and low compressibility to keep the buffer in place, and a low hydraulic conductivity. The grain size distribution of the backfill should be such that significant intrusion of the buffer into the backfill does not take place. Some swelling pressure (~100 kPa, corresponding to the weight of about four metres of loose rock) would be beneficial to support the tunnels against rock falls, to establish a tight backfill-rock contact and to counterbalance the effects of consolidation and piping in the backfill. Groundwater flow and transport of contaminants along tunnels and the excavation damaged zone (EDZ) around them could also be blocked by means of durable sealing structures. Buffering and sorption capacity for contaminants is a favourable property in the backfill material, too.

The desired properties of the sealing structures include

- no harmful effects on the other barriers
- chemical and mechanical stability over long time periods
- low hydraulic conductivity
- natural materials, like clay, with proven long-term properties are preferable to man-made materials.

A key question is: Needs the backfill to be homogenous and to have a swelling capacity and a hydraulic conductivity lower or equal to that of the surrounding rock? In our opinion, these desirable properties are not necessary in all parts of the backfill, provided that the transport pathways along the excavations and EDZ can be blocked by means of durable sealing structures. In this case, the repository would effectively be divided into isolated compartments, which internally may, initially or in the long term, have a (significantly) higher hydraulic conductivity than the virgin rock.

Alternatives for backfilling and sealings

Alternative backfilling and sealing structures currently under evaluation by Posiva include

- backfill mixture consisting of crushed rock and 10–30% of bentonite, and compacted in situ. It may be used in the upper parts of the access routes (swelling and non-sinking of the backfill in the fresh groundwater would, for example, ensure a support underneath for the upper sealing structures). However, it seems unlikely that a mixture of crushed rock and bentonite, compacted in situ, would function properly (i.e. have a swelling ability and low hydraulic conductivity) in saline groundwater with a TDS of 35 g/l (Karnland 1998, Dixon 2000). Therefore, this backfill is currently not planned to be used in the deposition tunnels of the repository. Furthermore, in an unlikely future situation where Olkiluoto would be flooded by ocean water, a backfill mixture of crushed rock and bentonite may neither function properly. There might be reasons to reconsider whether the backfill in all parts of the disposal facility really needs to withstand groundwater salinities up to 35 g/l in all postulated scenarios.

- natural mixed-layer clays which have a lower swelling ability than bentonite, and are less expensive. Friedland clay from the Neubrandenburg area in northern Germany, which has a clay content of about 90% of which about 50% are expandables (montmorillonite and mixed-layer smectite/mica) is studied in cooperation with SKB. In all key functions, its properties are evaluated to be equal or superior to those of a 30/70 mixture of bentonite and crushed rock (Pusch 1998, 1999, 2001). Furthermore, the performance of Friedland clay is not significantly affected by groundwater salinity. The compactability of the clay suggests that it can be applied in KBS-3 tunnels and shafts with a density corresponding to 1900 to 2100 kg/m^3 after saturation. For about 1900 kg/m^3 density the conductivity is less than 10^{-10} m/s and the swelling pressure somewhat more than 200 kPa when the salt content is 3.5%. If backfills consisting of the investigated clay can be applied with a density of 2050 kg/m^3 at complete water saturation the hydraulic conductivity is lower than 10^{-10} m/s even at salt contents of up to 20%, and the swelling pressure will be higher than 600 kPa (Pusch 2001). It remains to be demonstrated on a full scale that Friedland clay ground to a suitable grain size distribution can be sufficiently compacted in the repository conditions.

- A "compartmentised" repository where parts of the deposition tunnels are backfilled with crushed rock, and the transport pathways along the tunnels and in the excavation damaged zone (EDZ) are blocked by means of durable sealing structures of highly compacted bentonite. Design of this alternative is discussed in the next section. Special emphasis needs to be devoted to performance assessment of a repository, parts of which may, initially or in the long term, have a (significantly) higher hydraulic conductivity than the surrounding rock.

- The Medium Long Hole (MLH) concept (Autio et al. 1996), where canisters are emplaced in long horizontal deposition holes and are surrounded by highly compacted bentonite, would, in some sense, provide an ultimate solution to backfill issues, as in this concept there is only the buffer of highly compacted bentonite in the deposition rooms. However, backfilling and sealings are needed in the central tunnels, auxiliary rooms and access routes in this concept, too.

Different backfill materials and mixtures may be used in the different parts of the deposition tunnels, and in auxiliary rooms and access routes at different depth levels. From the practical and economical point of view, it is desirable that a major part of the disposal facility could be backfilled with a material manufactured from the rock excavated from the repository. This rock has the added advantage of being compatible with the ambient geochemical environment.

Plugs of highly compacted bentonite in tunnels backfilled with crushed rock

A preliminary design of a impermeable, durable plug in a deposition tunnel backfilled with crushed rock is shown in Figure 2. The plug consists of blocks of highly compacted bentonite. Concrete is not used in order to avoid potential harms due to cement-bentonite interaction. The development of low-alkaline cements may, however, enable to use also concrete in the construction of the sealing structures. Flow and transport along the EDZ is blocked by V-shaped slots. The size and shape of the slots and plugs have to be adjusted to the rock quality, support from backfilling, pre-closure hydraulic conditions and state of stress to obtain a stable structure and a break in the EDZ which is long enough to block the flow of groundwater effectively. Erosion and intrusion of compacted bentonite into the crushed rock may be prevented by a layer of filter material with an optimised grain-size distribution. A similar filter layer may be emplaced also on the top of the bentonite in the deposition hole.

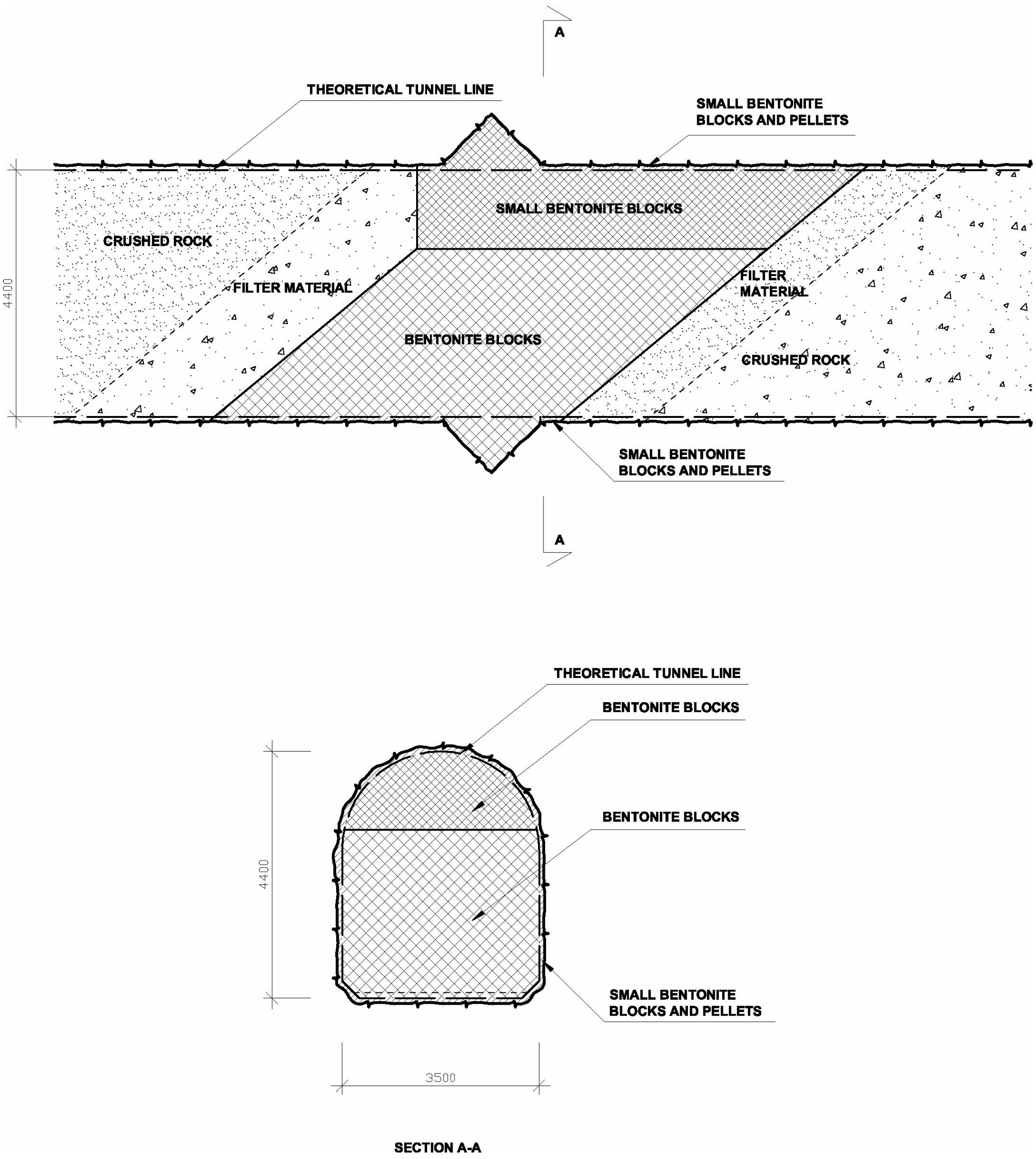

Figure 2. *A preliminary design of a plug of highly compacted bentonite in a tunnel backfilled with crushed rock.*

The size and dimensioning of the plugs is determined mainly by constructability, hydrostatic and other loads, and long-term durability and reliability. The loads exterted on the plugs, and the associated risk of erosion and piping in the bentonite, are highest during the operation phase when parts of the repository behind the plugs are open. The postclosure performance requires that the plugs maintain a sufficient density (around 2000 kg/m^3) to remain impermeable and resistant to saline groundwater.

Plugs need to be constructed at least at the entrance of each deposition tunnel. A plug, but less massive, could be constructed also in the dead-ends of the deposition tunnels as the hydraulic gradient in the rock around the end of the tunnel may be significantly increased due to the “conductor in insulation” effect (Vieno et al. 1992). For the sake of reliability, additional plugs may be constructed within the deposition tunnels. The location of these plugs can be optimised taking into account that the distance between conductive features in the “intact” rock (i.e. rock mass between the fracture zones included in the hydrogeological model) at the depth of 300-700 metres at Olkiluoto varies typically between 60 and 140 metres, whereas the rock sections between these conductive features have usually a very low (< 10^{-10} m/s) hydraulic conductivity (Äikäs et al. 1999). In the central tunnels and access routes plugs will be located taking into account the fracture zones intersecting these parts of the repository.

Discussion

Posiva is currently planning in detail the Underground Rock Characterisation Facility ONKALO and the investigations to be carried out to characterise suitable rock volumes for the repository. In this context, preliminary plans for the backfilling and sealing will be developed for ONKALO as well as the forthcoming repository. Backfilling issues currently under discussion include:

- Backfill mixture of crushed rock and bentonite: Would it be (significantly) inferior to crushed rock in case of intrusion of saline groundwater because of potential erosion and associated colloid generation of the bentonite fraction? Can it be used in the upper parts of the disposal facility (significance of the flooded-by-ocean scenario)?
- Friedland clay: technical feasibility, properties in the scale of the tunnel.
- Backfills including a clayey component: Is there a risk that groundwater leakages could cause significant problems for emplacement.
- Backfill of crushed rock with durable sealing structures of highly compacted bentonite
    - detailed design and emplacement/construction of the backfill and plugs
    - filter material to prevent intrusion and erosion of bentonite from the plug into the crushed rock
    - locating of the plugs
    - pre- and postclosure performance and durability of the plugs (high gradients during the operation phase, erosion and piping in the long term)
    - properties (hydraulic conductivity, compressibility) of crushed rock in the scale of the tunnel
    - performance assessment of a “compartmentised” repository.
- Backfilling and sealing of shafts and ramps: Is sinking of the backfill a potential problem?

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Paper 3

What are the Requirements of Sealing Performance?

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Japan Nuclear Cycle Development Institute (JNC)

Abstract

The sealing technology is considered to become the key items to succeed in the closure of the high-level radioactive waste (HLW) repository. The performance assessment will decide the requirements of the performance of the sealing technology for the HLW repository. The performance assessment is performed based on the scenario analysis of the multi-barrier system (the engineered barrier system and the geosphere).

Through the discussion of the requirements of the sealing technology considering the assumption of the performance assessment and the current civil engineering, we suggested the requirements of the sealing technology and their verification methods. The requirements of the sealing technology are categorized into those for a short- and a long-term. The engineering scale experiments are efficient to verify the performance of the sealing technology.

Key words; sealing technology, short- and long-term, civil engineering, performance assessment

Introduction

Sealing technology is important to close the HLW repository in the deep underground. The safety and feasibility of the disposal of the HLW are ensured by the performance assessment. The performance assessment is based on the FEP (Feature, Event, Process) analysis. In Japan, the FEP analysis assumes that the radionuclide will migrate from the engineered barrier system (EBS) to the biosphere through the geosphere [1]. Considering the performance assessment, there are various assumptions on the properties of each material of the repository. Therefore, the sealing technology has requirements of performance from the viewpoint of mechanics, hydrology and geochemistry.

Japan Nuclear Cycle Development Institute (JNC) has already performed some studies on the sealing technology so far. For example, Saotome et al. showed the hydraulic and radionuclide migration analysis along the shafts [2]. This analysis showed the effectiveness of the installation of plug, backfill and grout (see Figure-1). However, this analysis considered no alteration of the sealing materials. In addition, the long-term stability of clay-based grout injected into rock mass was assumed. This primitive analysis included significant uncertainties.

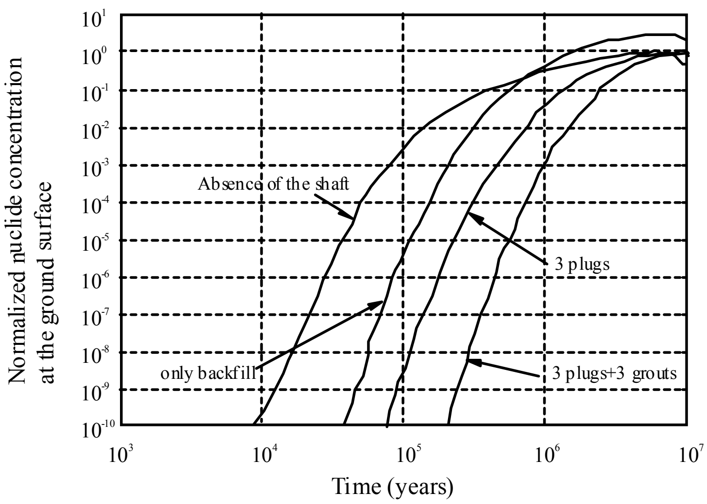

Figure-1 *Time dependency of normalised nuclide concentration [2]*

The performance assessment has to be performed considering the barrier performance demonstrated by the feasible civil engineering. However, it is not easy to cooperate closely between the civil engineering and the performance assessment. Therefore, very little has been studied on the performance assessment considering the civil engineering for the sealing technology.

Components of the sealing technology

JNC has published the H12 report to establish the scientific and technical basis for the HLW disposal in Japan [1][3][4]. The sealing components adopted in H12 report are backfilling material, plug and grout as shown in Figure-2 [3]. The backfilling material is the mixture of bentonite and aggregate. Large amount of aggregate will be manufactured in construction phase of the repository. The plugs are the concrete plug and the clay-based plug. The grout is clay-based material.

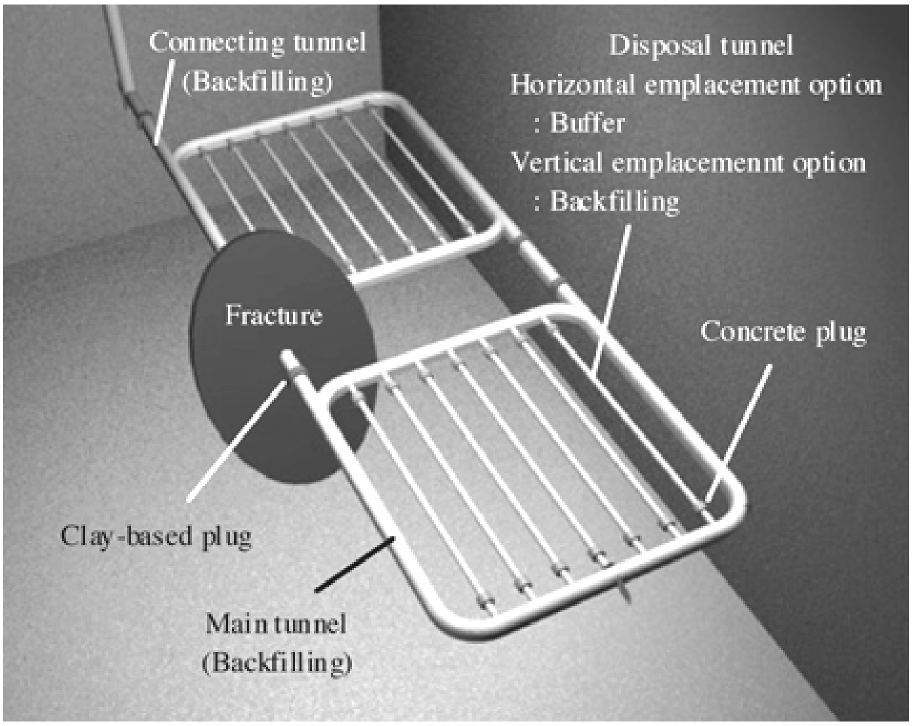

Figure –2. *The concept of backfilling of the HLW repository [3]*

Scenario analysis of the radionuclide migration

The EBS has design requirements, e.g. radionuclide retardation, buffer of the stress, self-sealing, mechanical support of the overpack, low hydraulic conductivity. These requirements rely on the buffer performance. The performance of the buffer mainly depends on the density. The decrease in the density of the buffer will lead to the decrease in performance e.g. radionuclide retardation. The buffer will be expected to maintain the designed density for the long-term. Therefore, after the EBS has been installed, the tunnels and shafts must be filled up and sealed without damaging the integrity of the system.

On the other hand, the geosphere is expected to have the requirements, e.g. radionuclide retardation, relevant hydraulic conductivity. It is considered that the long distance between the repository and the biosphere will be required.

In H12 report, JNC performed the performance assessment based on the assumption of the hypothetical repository as shown Figure-3 [1]. However, the realistic repository has many drifts and shafts that have to be filled up with backfilling material. So, we have to clarify what the requirements for the sealing technology are.

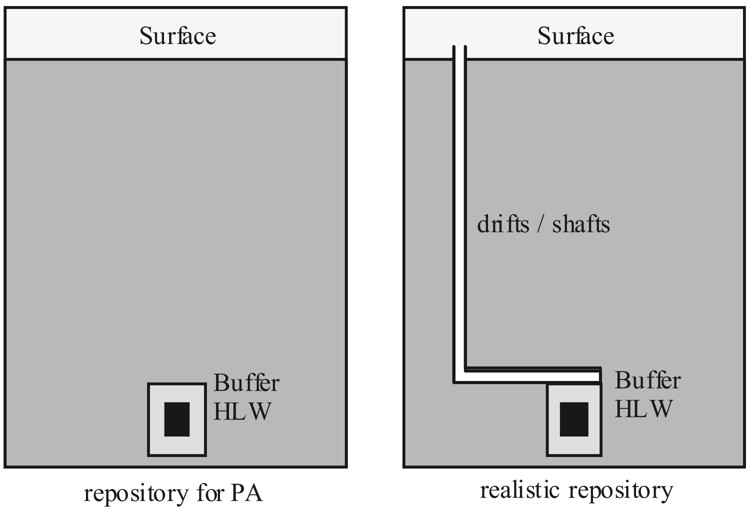

Figure-3. *Assumption of the repository for the performance assessment*

Requirements for the sealing technology

It is not so easy to analyse the performance of the realistic repository. That means the sealing technology should not disturb the assumptions in the performance assessment. In H12 report, we described that the backfilling is needed to eliminate the significant influences on the performance of the entire repository [3]. Pusch suggested the functions of the backfilling; restriction of the buffer extrusion, stabilisation of tunnels, and maintenance of low hydraulic conductivity with respect to the surrounding rock [5].

We tried to categorise the requirements of performance into a short- and a long-term. The short-term means the operation and closure phases, the long-term means the performance assessment phase after closure. We considered the requirements on the mechanical, hydraulic and geochemical performance of the sealing technology in each term. The focal sealing components are the plug and the backfill material (see Figure-4).

Figure-4 shows an example for consideration of the performance of the sealing technology, which indicates radionuclides pathways. The performance A shows that for a pathway through tunnels. The performance B shows that adopted in a pathway in H12 report. When the performance A is better than the performance B, the sealing technology does not disturb the assumption of the performance assessment adopted in H12 report. To maintain performance A better than performance B is the requirements of the sealing technology. The following items are the considering points of the each term performance.

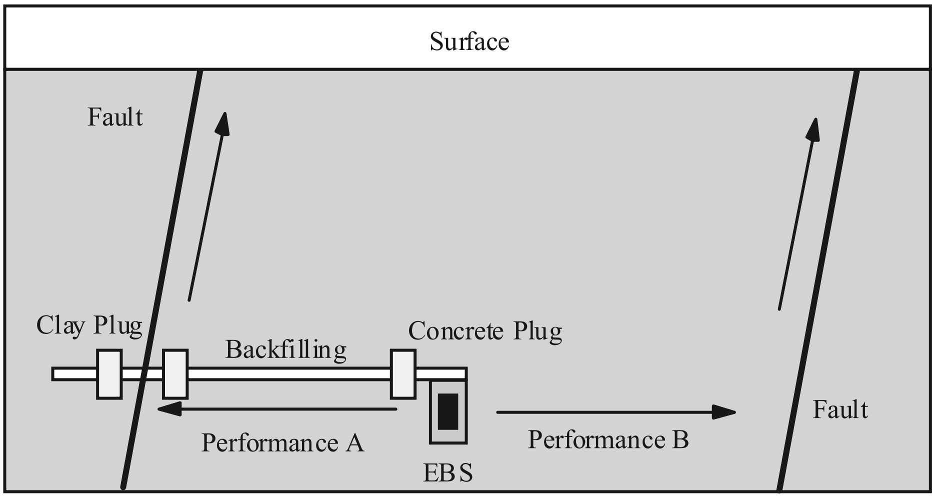

Figure-4. *An example for consideration of the performance of the sealing technology*

Short-term performance

-Backfilling material;

Mechanical stability (restriction of the swell of the buffer)

-Concrete Plug;

Mechanical stability (restriction of the swell of the buffer/backfill)

-Clay-based Plug;

Low hydraulic conductivity (isolation of the fractured zone)

Long-term performance

-Backfilling material;

Mechanical stability (restriction of the swell of the buffer)

Low hydraulic conductivity

Geochemical stability

-Concrete Plug

Geochemical stability (disturbance to other materials)

-Clay-based Plug;

Low hydraulic conductivity (isolation of the fractured zone)

Geochemical stability

Backfilling material needs to function the restriction of the swell of the buffer in the short- and the long-term to keep the designed density of the buffer, low hydraulic conductivity to keep the radionuclide retardation in the tunnel and the shaft in the long-term, and geochemical stability not to alter significantly buffer, rock and itself in the long term.

Concrete plug needs to function the restriction of the swell of the buffer in the horizontal emplacement option or of the backfilling material in the vertical emplacement option to keep the designed density of the buffer or backfilling material in the short-term, geochemical stability not to alter significantly buffer, backfilling, rock and itself in the long-term. For geochemical aspects, low-alkaline concrete is likely preferable under present situation of research progress.

Clay plug needs to function low hydraulic conductivity to isolate the fault or fractured zone expecting not to be the critical pathway through the tunnels in the short- and the long-term, and geochemical stability not to alter significantly buffer, backfilling, rock and itself in the long-term.

Verification of the performance of the sealing technology

JNC has performed the tunnel sealing experiment (TSX) at the URL in Canada with AECL, ANDRA and SNL [6]. This experiment focuses on the demonstration of the sealing technology. In the TSX, the full-scale concrete plug (low-alkaline, high performance) and clay-based plug were constructed at 420m level drift. There is a pressure chamber between both plugs.

Figure-5 shows the schematic view of the radionuclide migration pathway around the plug. The expected pathways are the geosphere (near-field host rock including EDZ), the EDZ, the interface between the EDZ and backfill, and backfill itself. We have to understand in detail which will be the critical pathway to show the safety and feasibility of the disposal of the HLW. Through the TSX, we have obtained the data to characterise the critical pathway in and around the plug. The TSX project will identify critical pathways in a short-term by sealing performance experiments.

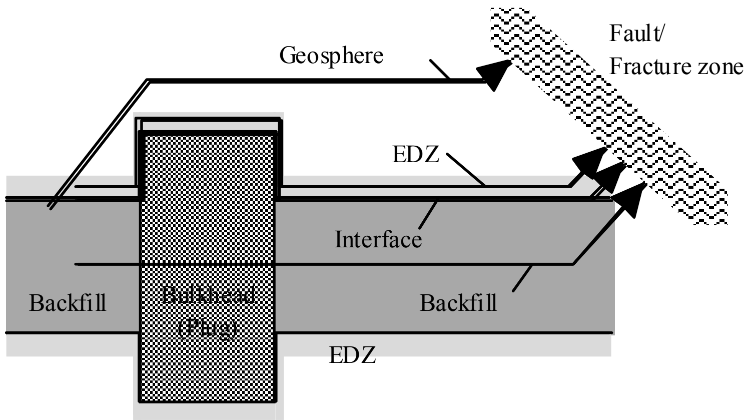

Figure-5. *Characterisation of the critical pathways*

JNC also has joined the prototype repository project (PRP) at Äspö in Sweden [7]. The PRP focuses on the understanding of the systematic behaviour of the EBS including backfill of the experiment drift. Therefore, the PRP will also provide the construction technology of the designed backfilling material and the concrete plugs. Many sensors will be installed in the backfilling material to measure the sealing performance. Eventually, the PRP will provide the civil engineering on emplacement of the backfill, and the data regarding barrier performance of the backfill in the short-term.

Conclusions

The requirements for the sealing technology have been discussed. We focused on the FEP analysis and performance of the sealing technology. We have to understand the difficulty of the closely cooperation between the civil engineering and the performance assessment. And data exchange and close linkage between civil engineering and the performance assessment are important to assess an actual repository.

We think the requirements of the sealing technology should be categorized into those for the short- and the long-term. The considered components of the sealing technology are backfilling material and plugs. The performance of the sealing technology should be considered from the viewpoint of mechanical, hydraulic and geochemical performance.

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Paper 4

A new method for tunnel backfilling

Tapani Kukkola, Fortum Nuclear Services Ltd

Workshop on Backfilling Requirements in a KBS-type Repository, Äspö Hard Rock Laboratory, August 27–28, 2001

The new idea: Tunnel backfilling with crushed rock, consolidated and tightened by injected cement or by other low-viscose material

The final disposal tunnels are filled with crushed rock up to the tunnel roof with help, for example, of a belt-thrower. The injection pipes are installed before the tunnel is filled. After filling of the tunnel at the lenght of one or two final disposal holes the liquid cement is injected into the crushed rock in order to tighten and to consolidate the backfilling. The crushed rock acts as backpressure during the cement injection. This will guarantee that the seams of the backfilling and the tunnel roof and the walls will be tight. The aimed advantages of this filling method are the efficiency in material logistics and the economy of the method. Also the possibility to avoid the plugs at the tunnel entrance is one of the aims.

Requirements and functions of tunnel backfilling

The main function of the backfilling is to prevent the disposal tunnels and access routes to the repository to become major conductors of groundwater and transport pathways of contaminants, and to block inadvertent intrusion into the repository. In a KBS-3 repository the functions of the backfill in the upper part of the deposition hole and in the tunnel further include (Vieno et al):

- To keep the buffer around the canister in place by counteracting the swelling of the buffer
- To contribute to keeping the tunnels mechanically stable.

The backfill should have chemically and mechanically favorable and predictable properties over long time periods and it should not have any properties that could significantly impair the function of the other barriers. The desired properties include a sufficient density and a low compressibility for keeping the buffer in place, and a low hydraulic conductivity. The grain size distribution of the backfill should be such that significant intrusion of the buffer into the backfill does not take place. Some swelling pressure would be beneficial for to support the tunnels against rock falls, to establish a tight backfill-rock contact and to counterbalance the effects of consolidation and piping in the backfill. Buffering and sorption capacity for contaminants is a favourable property for the backfill material, too.

Requirements of logistic and economy for the backfilling

The tunnel backfilling shall also meet the needs of material logistics and economy, which are, for example:

- At least a distance of two final disposal holes, i.e. 8 meters in tunnel, shall be filled in one working shift. The tunnel filling of 18 meters should be the target of the efficiency.
- The tunnel filling material shall be able to be handled mechanically. The filling machinery can consist only of a few machines because lack of space in the final disposal tunnel. Existing machinery is preferred to prototypes.
- The tunnel filling shall be economically feasible. The cost target for the installed filling should be less than, say 200 €/m^3.

The reinforced concrete plugs at the entrance of the final disposal tunnel extend the length of the disposal tunnels. The plugs are also expensive and time consuming to construct.

Tunnel filling technique shall be such that it guarantees the good filling quality with a few quality control activities.

Advantages of the new backfilling method

A novel backfilling technique where a low-viscose material is injected through the preinstalled tubes into the crushed rock has been investigated. The method is expected to provide a practical and efficient way to produce a homogenous and tight mixture. The crushed rock acts as backpressure for the injection. This will guarantee that the interfaces of the tunnel filling and the rock will be tightly filled.

The tunnel backfilling with crushed rock, consolidated and tightened by injected cement or other low-viscose material, will evidently meet requirements of the material logistics and economy.

If the tunnels are filled with crushed rock and consolidated to a homogenous structure by the cement injection, the plugs are not needed anymore.

Drawbacks of new backfilling method

In the preliminary small-scale experiments, cement has been used as the injected material. However, cement may not be a suitable material to be injected into the deposition tunnels, for several reasons:

- Ordinary cement is an undesired material in the deposition area (low-alkaline cement might, however, be more suitable)
- Shrinkage of the backfill mixture could create open channels at the backfill-rock interface
- Gradual degradation of the cement is inevitable.

Injection of cement into the crushed rock may, however, be used for the construction of the sealing structures in other parts of the repository. Furthermore, the technique may be used for the injection of bentonite slurry into the crushed rock in cases where such a backfill is applicable.

It is not yet determined what is the acceptable amount of cement in the final repository spaces. The excess amount of cement may deteriorate the expansion capability of the compacted bentonite in the canister disposal holes. The low-alkaline cement may be the solution to the problem.

The long-term durability of a cement injection is not yet proved. Cement is an unstable material, which inevitable will weather with time. Additional clarification is needed to prove the long-term stability of the cement injection.

Mini-scale tests

Simple mini-scale tests for tunnel backfilling were made in summer 2001 (Kukkola 2001). The basic idea is to consolidate and to tighten the crushed rock used as the tunnel backfilling by using a cement injection. The first test arrangement is shown in Figure 1.

Figure 1. *The first mini-scale test arrangement.*

The intention of tests was to get an idea if the injection is a feasible technique for consolidation. 1.6 liter's cans simulated the tunnel; mixtures of sand and gritting aggregate simulated the crushed rock and the injection pressure was created by gravity. Sand grain size varies of 0.5 – 1.2 mm and gritting aggregate of 3 – 6 mm. Standard cement was used as an injection material.

Result of test is shown in Figure 2. The penetration was good, when the can was filled with gritting aggregate. For other tests the liquid cement injection pressure of two meters was too low. The air bubbles cause roughness on the upper surface of the test piece. Therefore it is necessary to get rid of air.

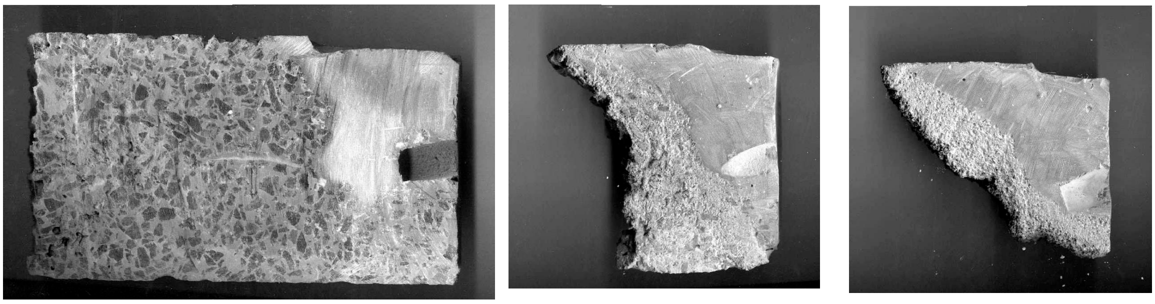

Figure 2. *The shear planes of the test samples.*

The 105-cm long 75-mm diameter plastic sewage pipe filled with gritting aggregate was used in the second mini-scale test. The test arrangement is shown in Figure 2.

Figure 3. *The second mini-scale test arrangement.*

The injection pressure was increased up to 3 meters. The test result is shown in Figure 4. The surface of the test piece is smooth; also the plane of fracture was without any cavities.

Figure 4. *The second test sample.*

Air can be removed by the effect of the liquid flow. A good contact and tightness with the formwork tube can be achieved. Based on these tests the technique seems to be feasible and worth for further design and testing.

Calculation analysis

Numerical method for estimating the flow of the injected cement would be a useful addition to the experimental studies. For example the optimization of the number and the locations of the injection places could be done. Preliminary analysis has shown that FLUENT computational fluid dynamics code may be used for the estimation of injected cement flow in the crushed rock (Toppila 2001). Figure 5 shows one calculation example of liquid cement flow in porous media.

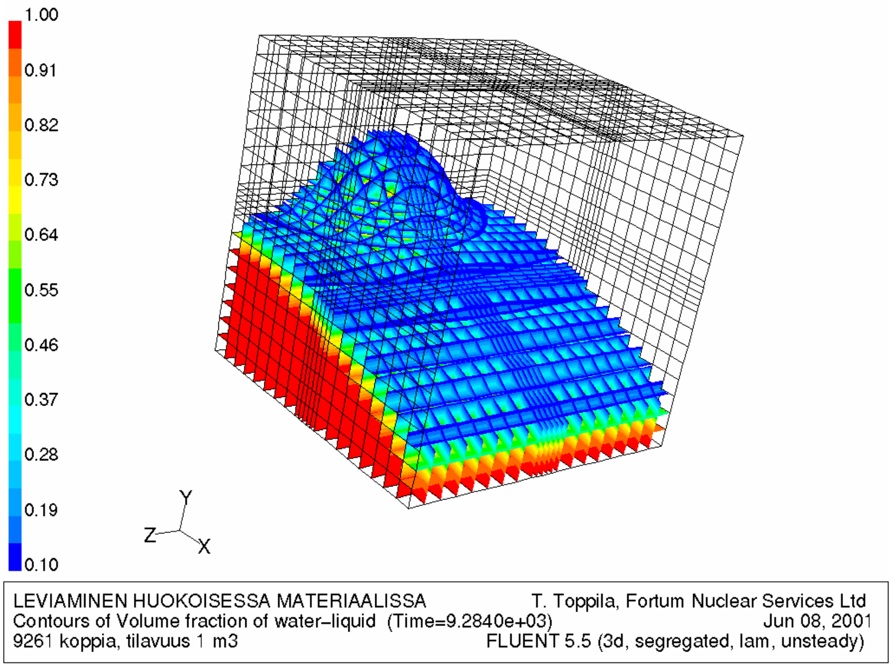

Figure 5. *The result of one calculation example.*

The idea of computation is that by using a numerical computation method the injection of cement can be optimized. It can also be studied that the tunnel will be completely tightened without any cavities in different tunnel geometry.

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Paper 5

Techniques for applications of low permeable soils

Heinz Thurner

Figure 1

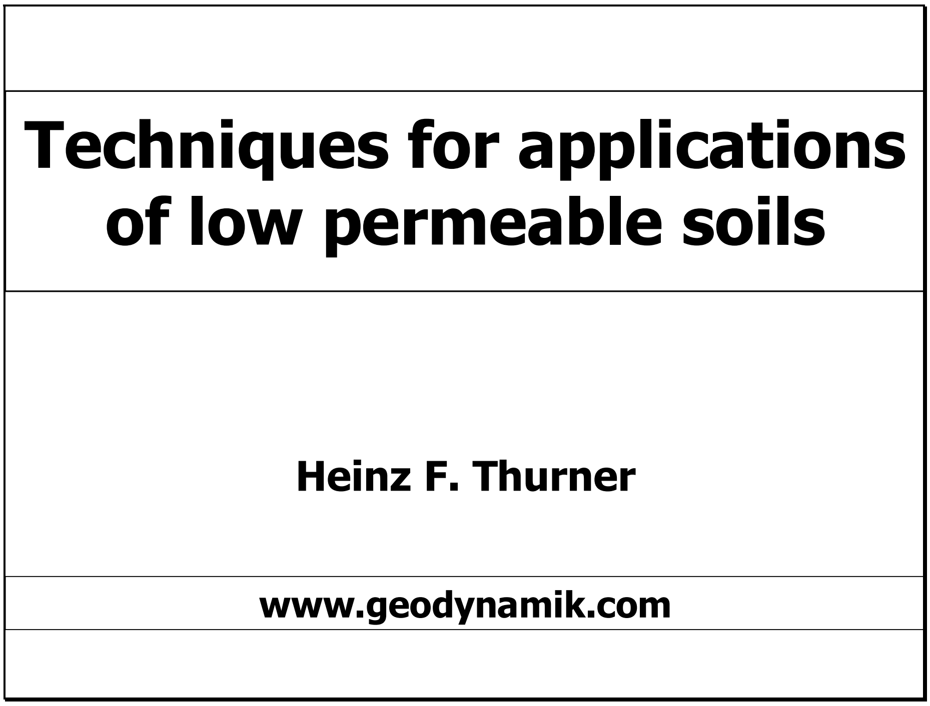

Figure 2

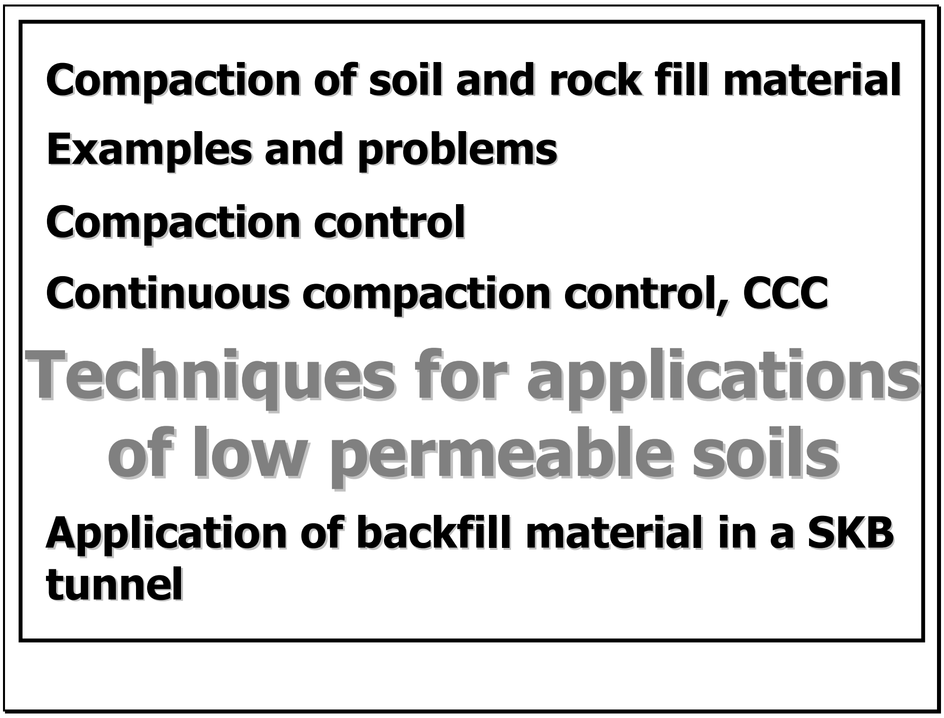

Figure 3

Compaction tools

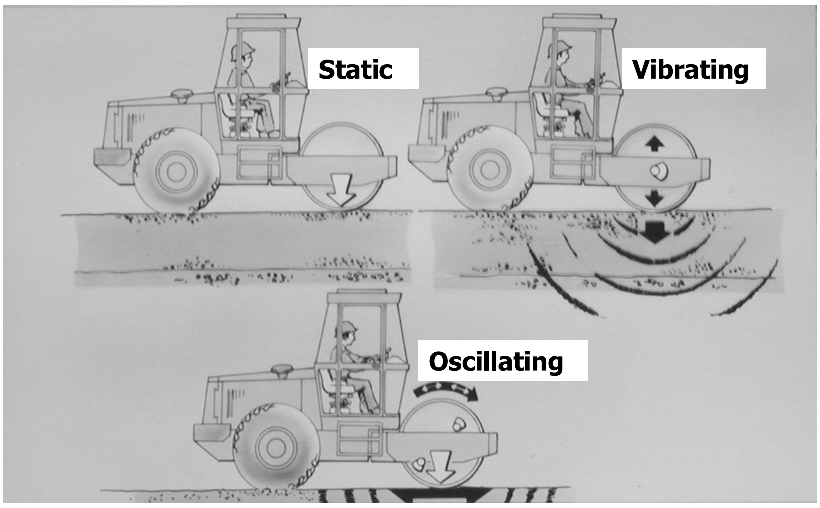

Figure 4

Problems
with compaction of fine grained materials

Liquefaction

Homogeneity

Settlement differences due to weak spots/incompact able spots

Figure 5

Compaction of soil and rock fill material

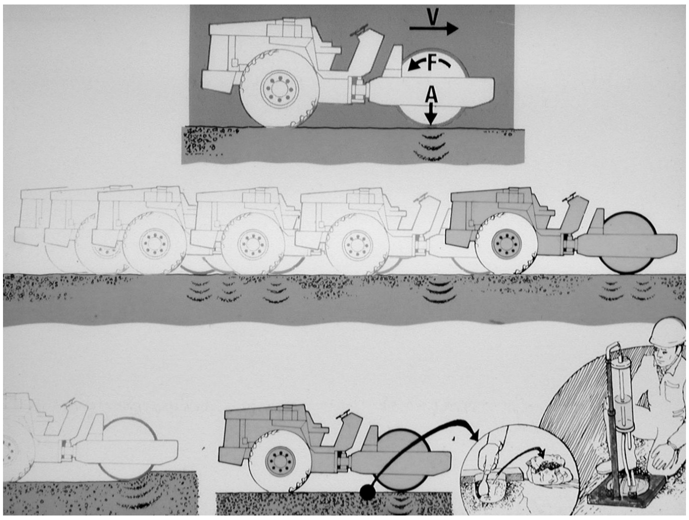

Figure 6

Compaction control

Spot test methods

- Volymeter
- Penetrometer
- Radiometric sond
- Static plate load test
- Dynamic plate load test
- FWDM

Continuous Compaction Control, CCC

- Compaction device = measuring device
- CMV/OMV
  - Dimensionless, relative value
  - Integral value (depth range, underground)
  - Instant, continuous value, repeatable value

Figure 7

Continuous Compaction Control
CCC

Compaction device = measuring device

CMV

- **Dimensionless relative value**
- **Integral value**
  - **Depth range**
  - **Underground**
- **Instant and continuous value**
- **Repeatable value**

Figure 8

Application of backfill in SKB-tunnels

Backfill material

Compaction device

Operator information

Continuous compaction control

Documentation

Figure 9

Backfill material

Uniformity

- Grain size distribution
- Water content

Emplacement

- Layer thickness
- Surface inclination

Figure 10

Compaction device

Carrier

- Joy stick
- F/A-adjustment
- Positioning
- Display

Slope compactor

- Vibrating (oscillating)
- Accelerometers
- E-sensor
- 3D-positioning

Roof compactor

- Vibrating
- Accelerometers
- E-sensor
- 3D-positioning

Figure 11

Operator information

Real time presentation on display

- **CMV/OMV**
- **Frequency and amplitude**
- **Surface inclination**

Compaction status

- **Compaction accomplished**
- **Additional blows required**
- **Avoiding of over compaction**
- **Location of incompactable spots**

Test procedure

- **Standardised parameters**
- **Entire area**
- **Acceptable and not acceptable zones**

Final blows for documentation

Figure 12

Continuous compaction control

Compacting tool = measuring device

Each blow recorded and analysed

Real time presentation to the operator

Test procedure

Documentation

Figure 13

Documentation

Standardized parameters

3D-Graph

- **Compaction accomplished**
- **Insufficient compaction**
- **Integration of CMV and spot tests**

Paper 6

General aspects of groundwater flow in the EDZ

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Introduction

This paper summarise some general aspects, which are mainly viewed from the ZEDEX study (e.g. Olsson et al., 1996) at the Äspö HRL, on groundwater flow in the excavation disturbed zone (EDZ). However, some experiences from the Stripa Mine, Buffer Mass Test (e.g. Pusch & Stanfors, 1992) are also considered. Further some general ideas about the problem an EDZ may give rise to and how these problems are related to time and scale are discussed.

The groundwater flow is governed by the hydraulic conductivity of the medium and the ambient hydraulic gradient. In a hydrogeological perspective, initial effects of the EDZ on the medium are mainly due to induced changes in the hydraulic conductivity caused by blasting and/or stress redistributions.

The ZEDEX study

ANDRA, UK Nirex and SKB performed the ZEDEX study at the Äspö HRL as a joint project, between April 1994 and June 1995. The study included investigations prior, during and post excavation of two drifts. A traditional drill & blast method was used to excavate one drift and the other drift was excavated by means of tunnel bore machine (TBM).

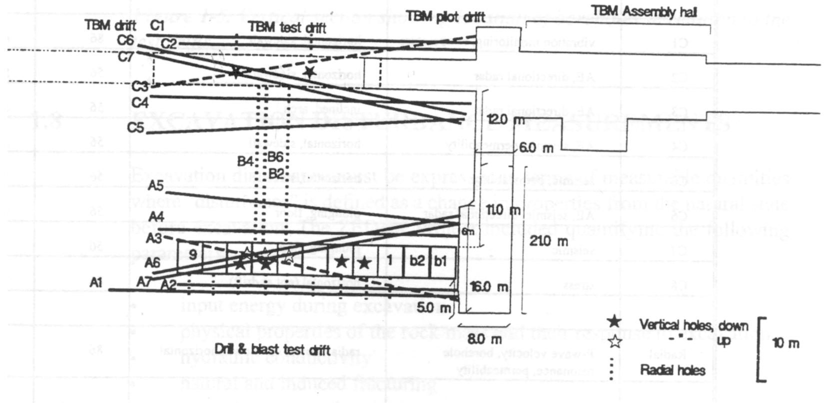

__Figure 1__ Horizontal section showing the location of boreholes in relationship to the two test drifts (taken from Olsson et al., 1996).

The initial main principal stress is estimated to be approximately 32 MPa and horizontal, the other two are estimated to be 17 and 10 MPa, respectively, the latter approximately vertical. The study did not explicitly include as a major aim to investigate the hydraulics surrounding the excavation, however, one is able to conclude on some important remarks on the hydraulics from the various tests done within the study.

Observations

Regarding the results from the ZEDEX study no clear evidence of any kind of increased or decreased hydraulic conductivity due to an EDZ could be observed. However, in view of the EDZ it is concluded that the damaged zone is significantly shallower in the TBM case compared to the drilled and blasted excavation. But, the water-yielding capacity could be both clearly higher in the periphery but also in other radial boreholes no peripheral differences compared to the host rock could be observed.

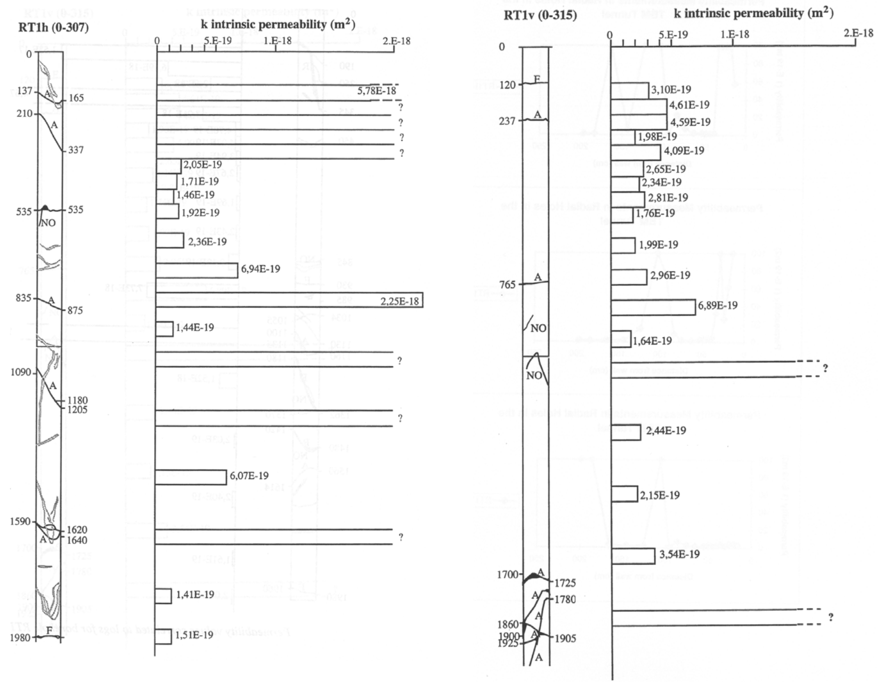

__Figure 2.__ Evaluated permeability values for two radial boreholes extending from the TBM drift (taken from Olsson et al., 1996).

In the ZEDEX study, the pulse tests in the radial boreholes were performed on a centimetre scale. These radial boreholes are divided into horizontal, vertical and inclined boreholes and do in general give the impression of an increased permeability zone of 0 – 50 centimetres from the excavation wall. This high permeability zone is especially clear for the inclined and vertical boreholes. These results are all post excavation and no comparison to pre-excavation data is possible. Some of the core loggings illustrate crushed or more fractured peripheral parts but other loggings do in principal illustrate the opposite.

Similar behaviour for results from both kinds of excavation techniques is evident. Boreholes drilled parallel (axial) with the drifts, shows the same kind of ambivalent behaviour with both increases and decreases of inflow to the borehole.

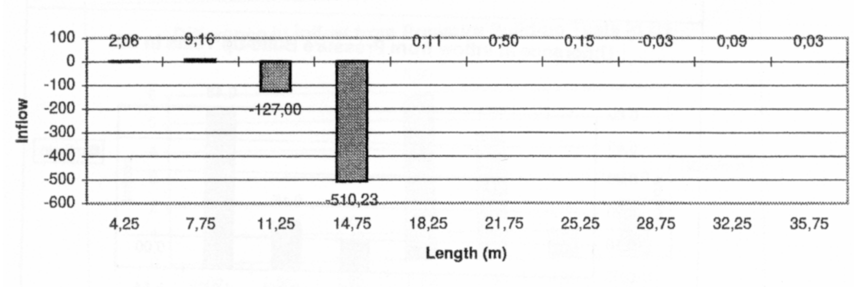

Figure 3. *Measured difference in ml/min. (before - after excavation) in inflow to borehole A5 sub-parallel with the drifts and approximately 4 - 7 metres from the drill and blast drift (taken from Olsson et al., 1996).*

These latter investigations were performed on a metre scale.

Previously conducted EDZ studies, e.g. in the Stripa Mine project show clearer evidence for increases in axial hydraulic conductivity. The Buffert Mass Test at Stripa (e.g. Pusch, 1989) concluded that the expected pressure build-up in the backfill material of a sealed-of part of a drift was not as large as expected, this even though the backfill material did reach the expected saturation rates. By measurement of the flow rates below the excavation floor it was concluded that in a zone of 0.5-1 metre the hydraulic conductivity had an apparent increase of hydraulic conductivity by a factor as high as 1000 times. Through this part of the floor a significant part of the groundwater was drained of to nearby-located parts of the mine with atmospheric pressure.

Further by measurements of the saturation in the backfill it was clear that the significant inflow of water into the sealed-of excavation had come from the lower parts of the excavation, it is congruent with the redistributed stress field that water should be coming from the excavation walls and not from the crown. If any water found its way through the floor into the sealed-of section is not evident, however possible.

Discussion

Firstly, an important remark is that for most of the ZEDEX study, no relationship with fringing structures is reported. In relationship to the application of the backfill material one water-yielding structure can cause significant problems.

In a short time perspective an EDZ may, due to its much higher hydraulic conductivity be drained of water much quicker than the surrounding rock yields water to the EDZ. This may cause an unsaturated situation within the EDZ (Ericsson, pers. com.). Such a condition is likely to decrease the hydraulic conductivity and together with the high tangential stresses reduce the inflow of water to the backfill material. A reduction of inflow such as this may inhibit a homogeneous wetting of the backfill material. Such unsaturation of the rock mass may further change the natural flow directions. This may be one likely explanation of the lateral flow (Ericsson, pers. com.) that was detected above the crown in some tracer experiments at the Stipa mine.

Additional in an unsaturated zone the pore water pressures are negative hence affecting the competence of the rock mass. This can lead to lesser convergence of the rock mass towards the excavation and hence limit the increase in the tangential hydraulic conductivity.

Thermal loads from the decaying waste are for a while also significantly higher than any swelling pressure (Alm, pers. com.) and these loads may for a time also inhibit a good wetting of the backfill; due to significant reduction of the radial hydraulic conductivity.

In a lengthier time perspective, when the system has returned to more natural pressures, more regional forces govern the groundwater flow. In the present state of knowledge, no future climatic situation is believed to cause a flow situation yielding a critical state in a one-way, high conductive pathway such as an EDZ would be if the sealing property of the plugs disappears.

Further, self-sealing due to e.g. calcite deposition are highly relevant for this subject.

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Ericsson. Professor L.O. Ericsson, Chalmers University of Technology, Department of Geology, Göteborg, Sweden.

Alm. Dr. P. Alm. VBB Viak AB, Göteborg, Sweden.

Paper 7

The role of backfill in suppressing brittle failure

Derek Martin

Figure 1

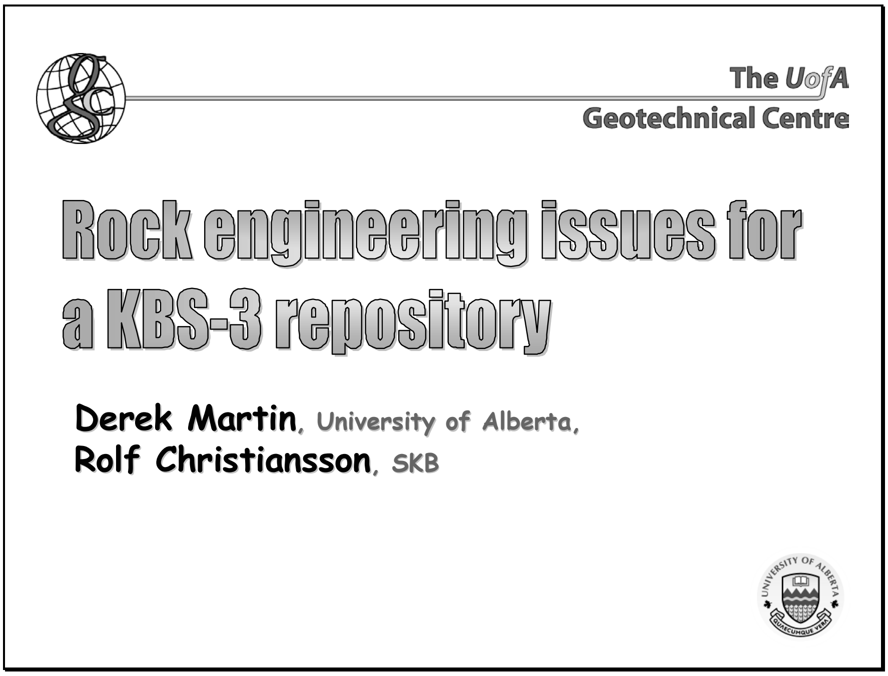


Figure 2

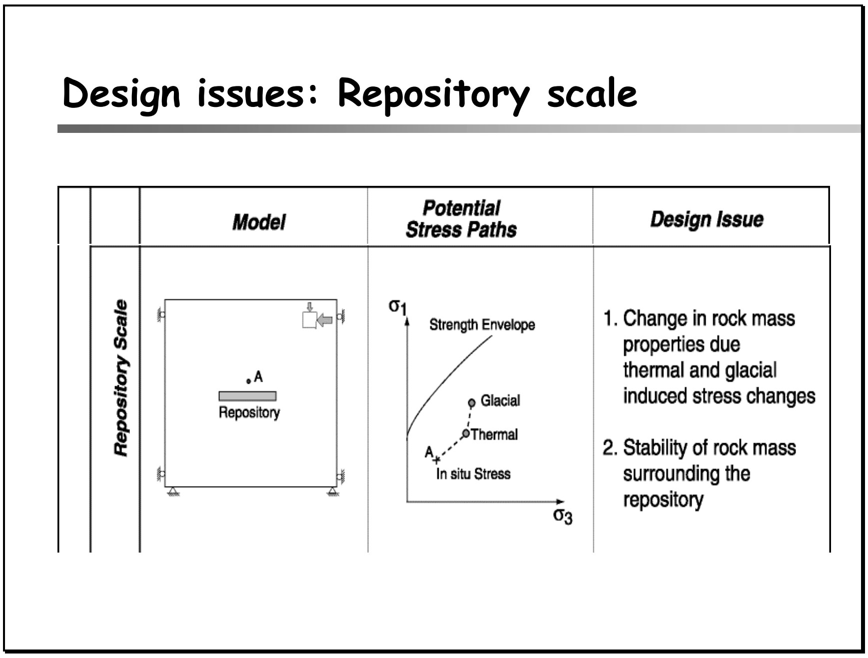


Figure 3

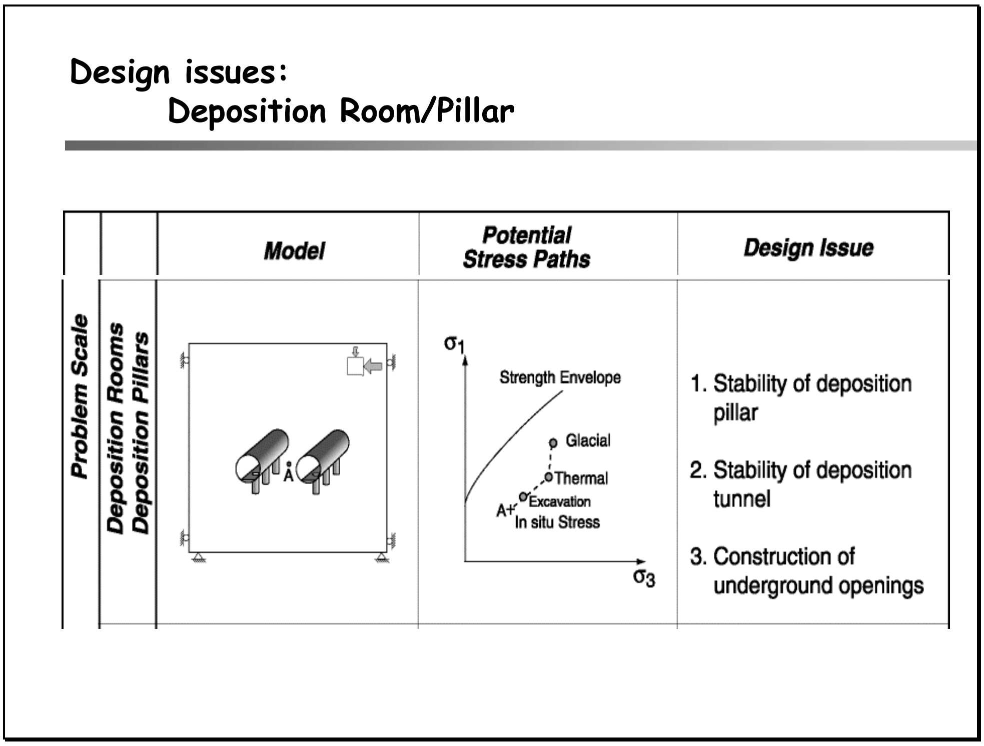


Figure 4

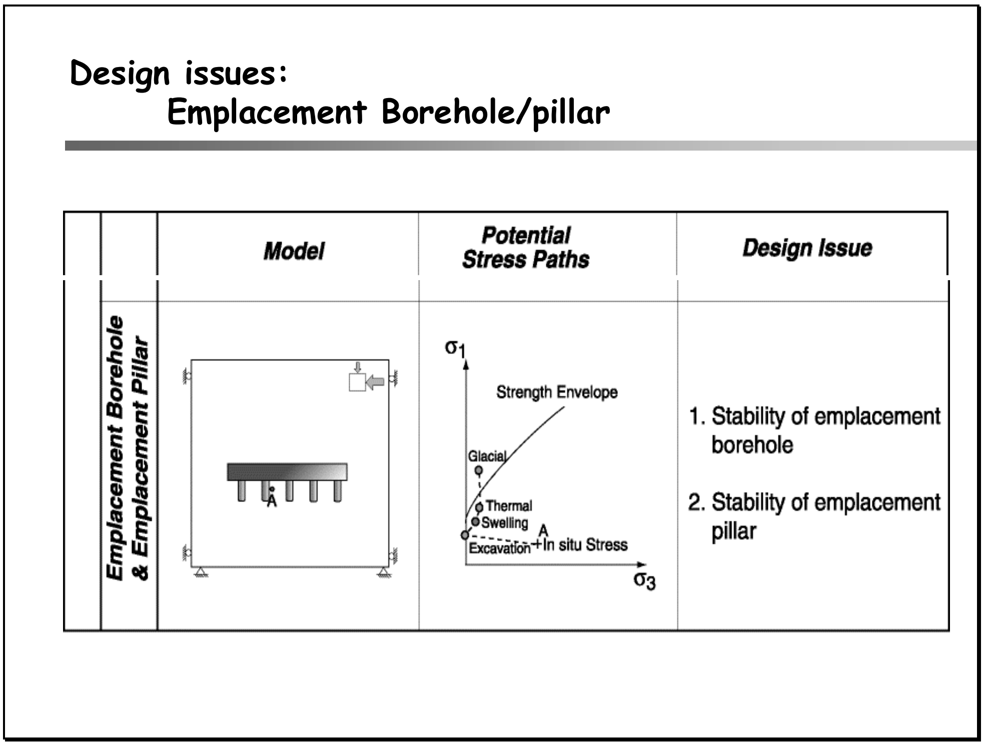


Figure 5

Failure modes

- Structurally controlled gravity-driven processes leading to wedge type failures
- Stress-induced failure causing slabbing and spalling
- Combination of structurally controlled gravity-driven processes and stress-induced failure.

Figure 6

Stress Path

***Note*:**
All failure processes around u/g openings are associated with low confinement

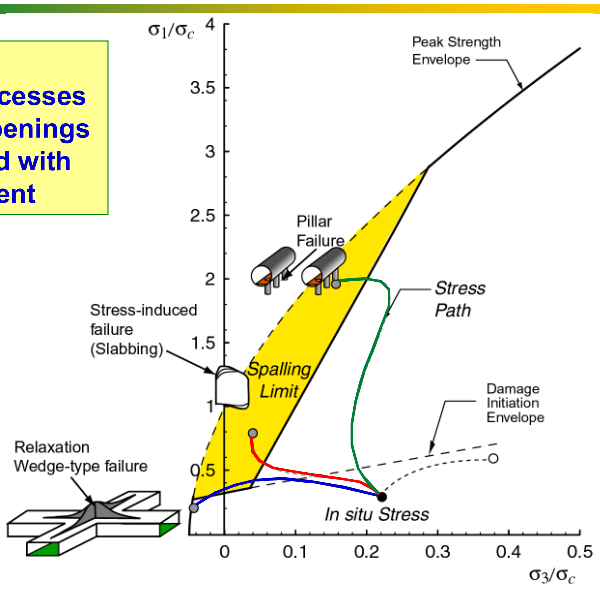
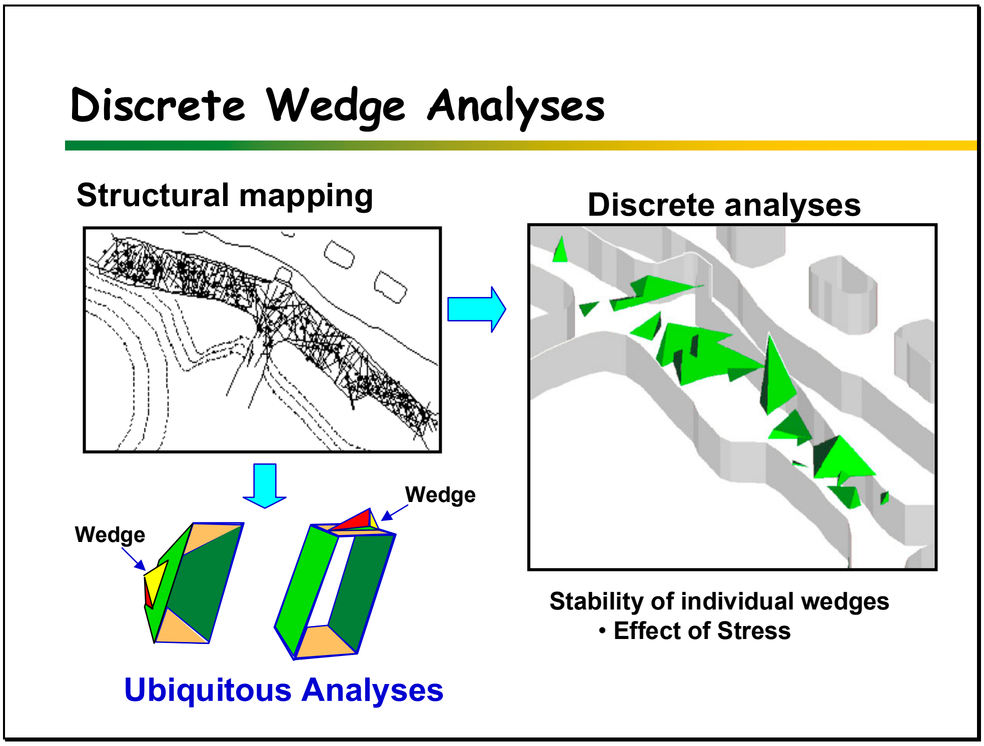


Figure 7



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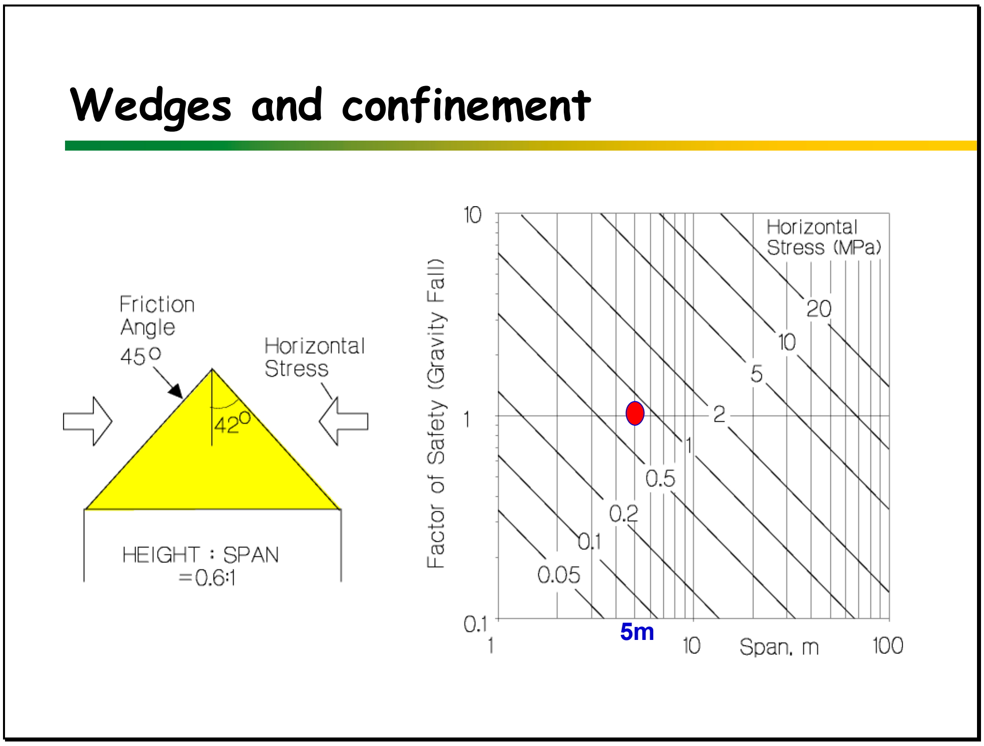

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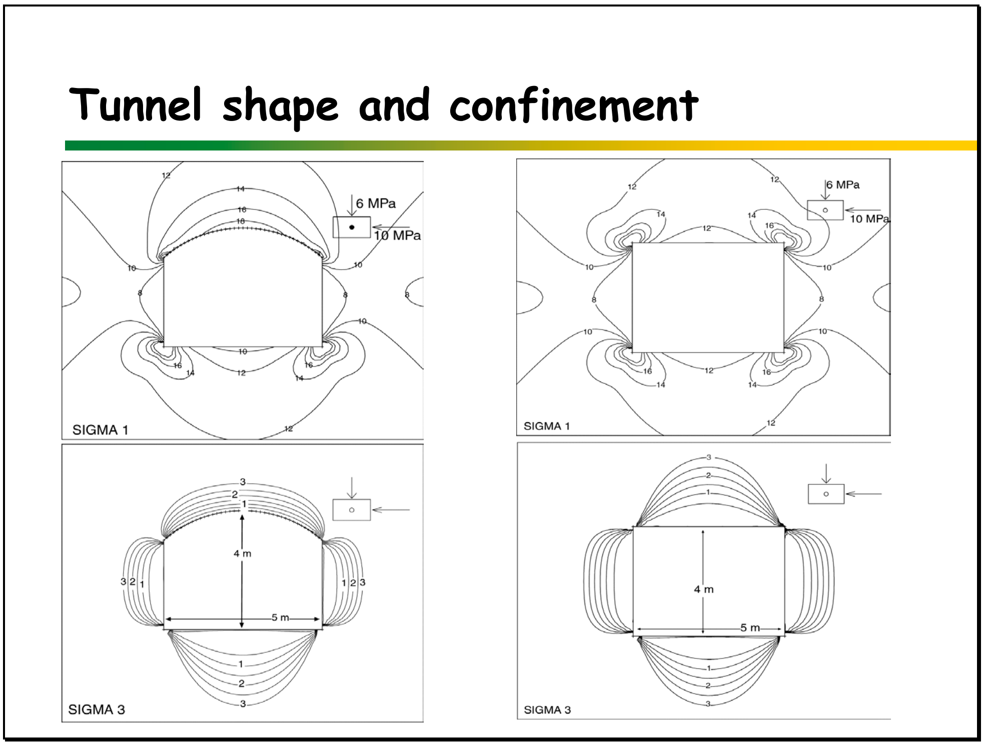


Figure 10

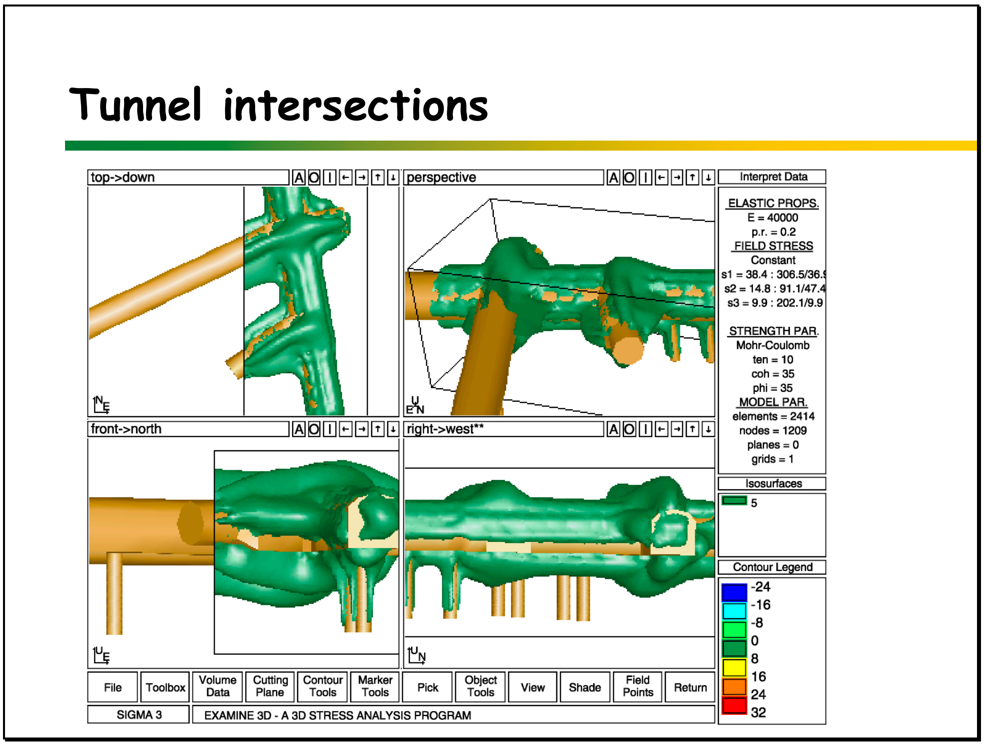


Figure 11

After Backfilling

- Wedges can only become an issue if there is kinematic freedom for movement
- After backfilling this kinematic freedom is removed.

Figure 12

Structure/Stress: Zedex

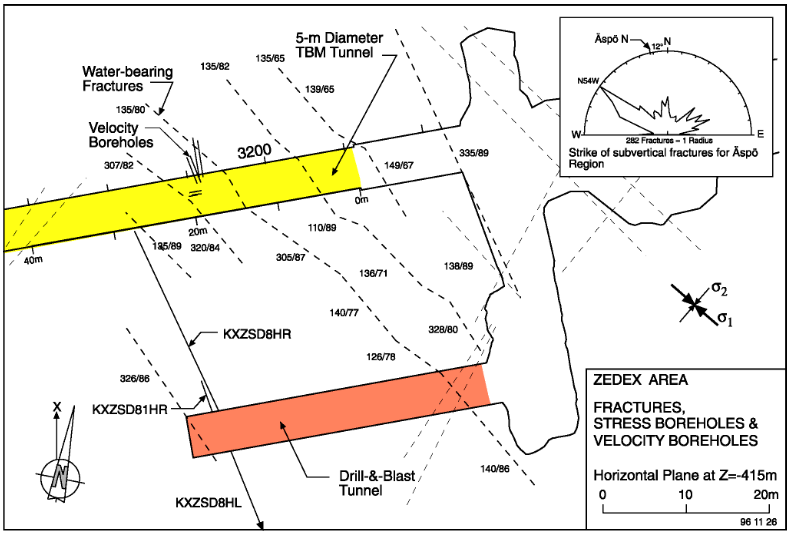
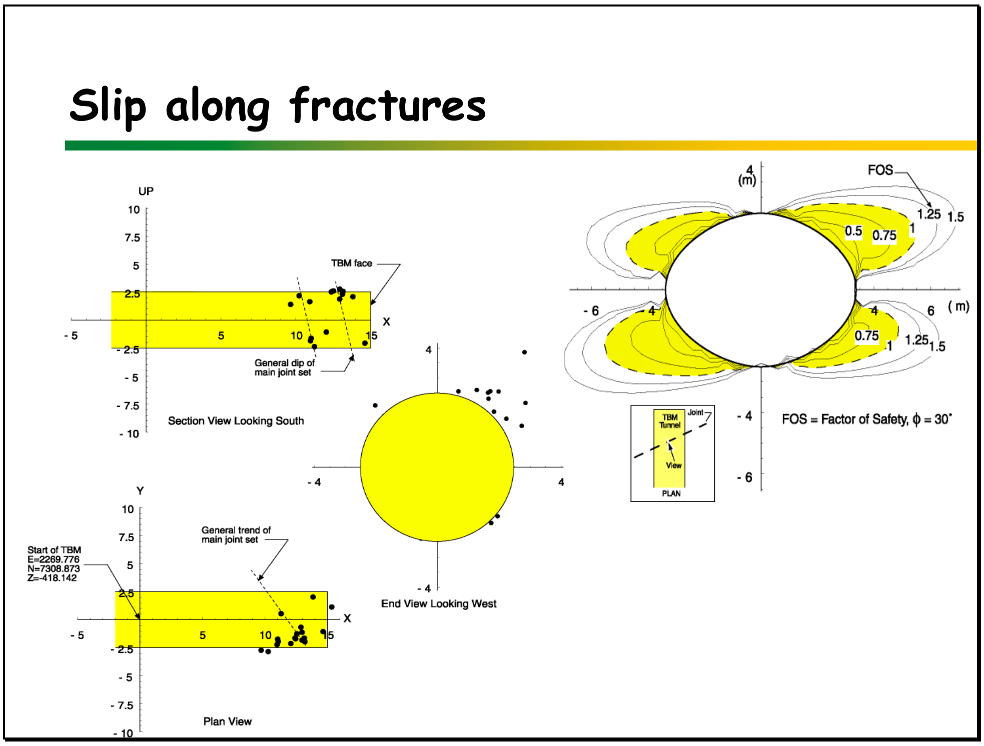


Figure 13



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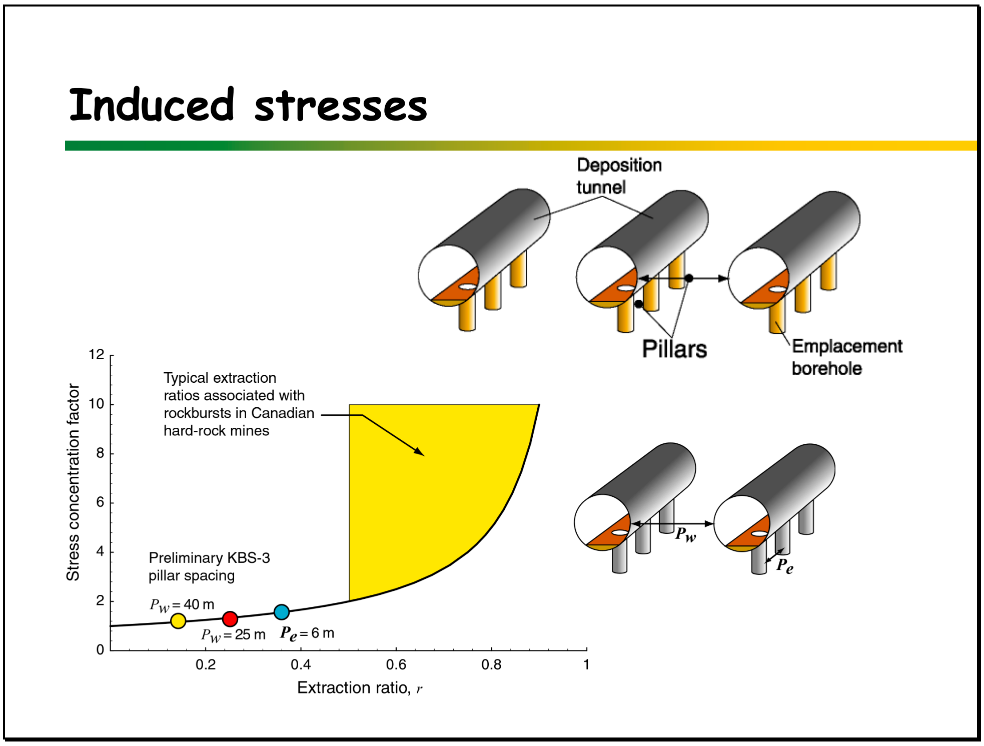


Figure 15

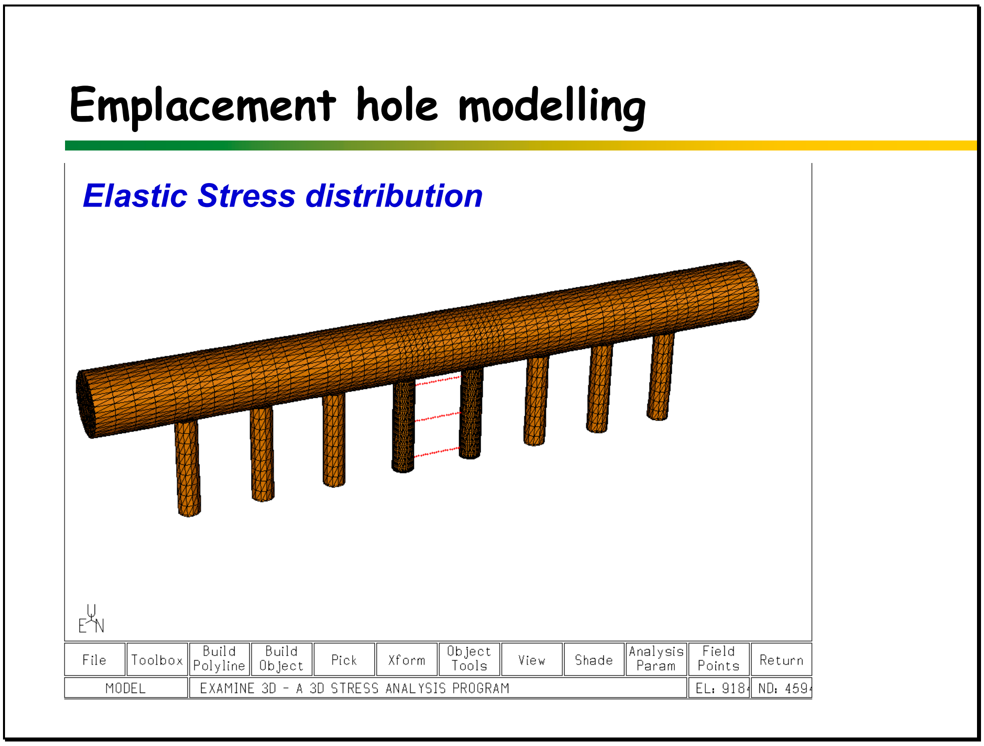


Figure 16

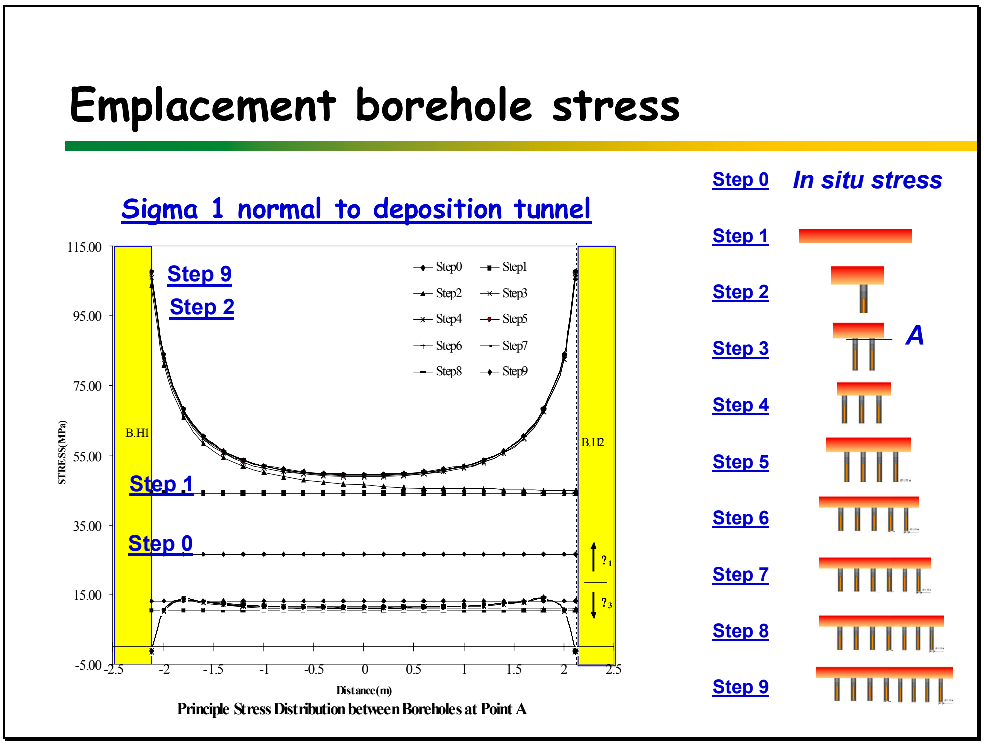


Figure 17

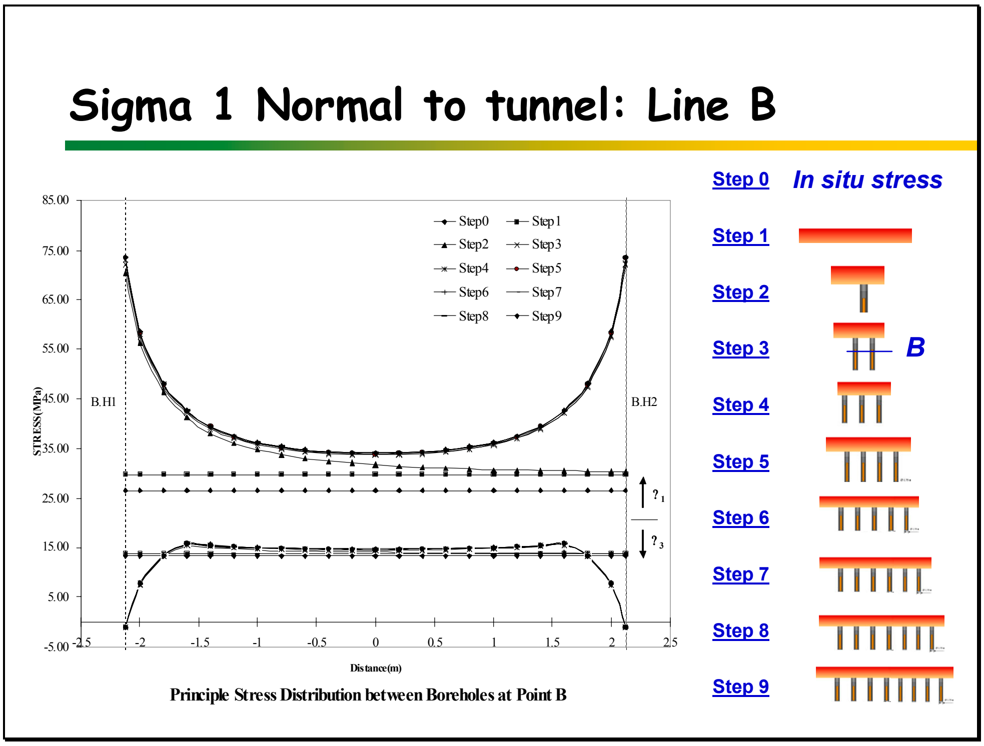


Figure 18

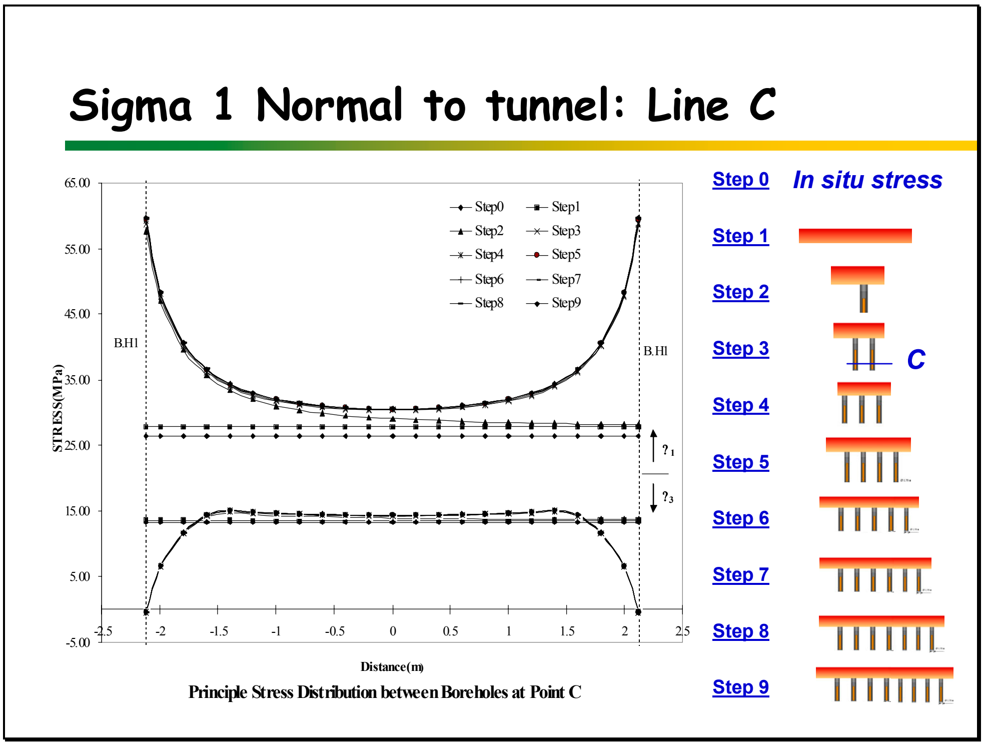
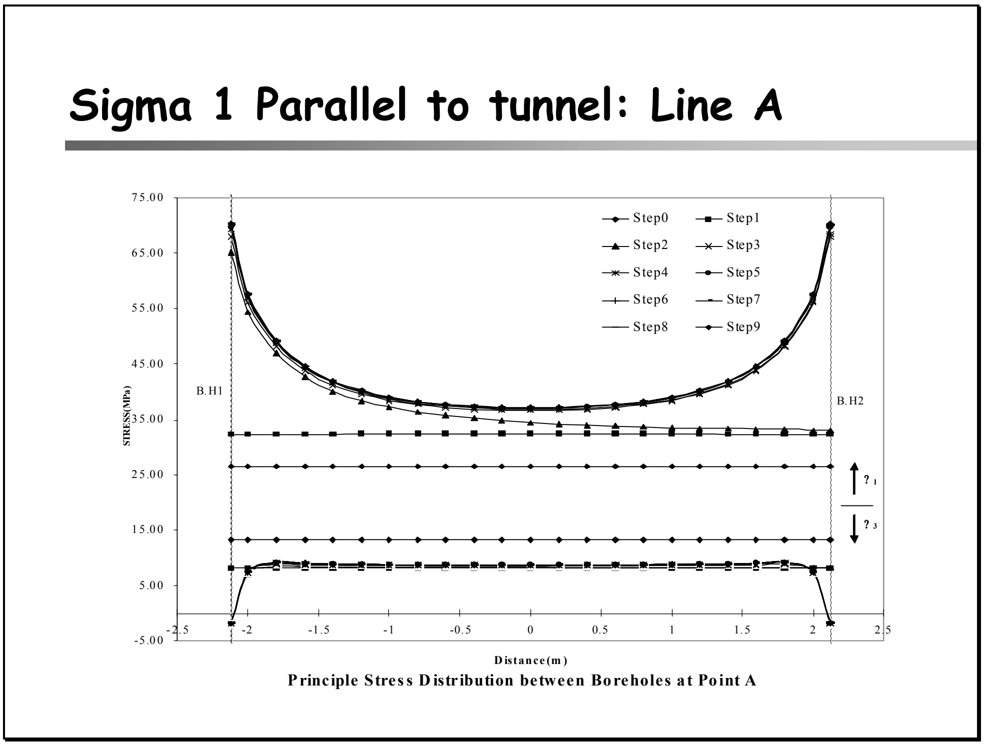


Figure 19



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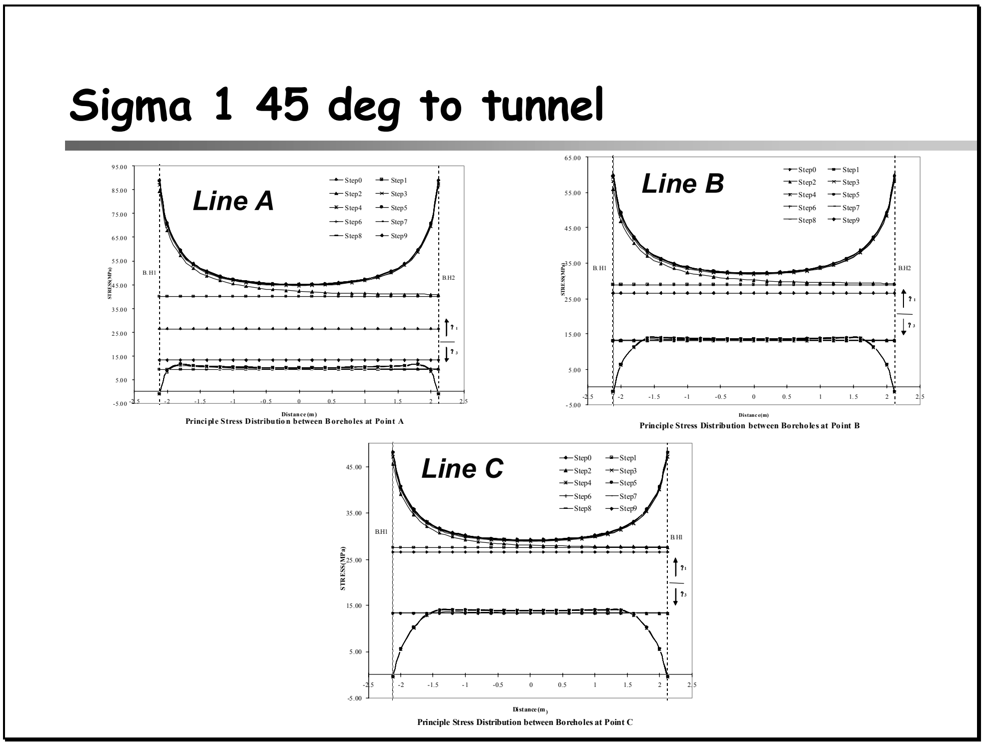


Figure 21

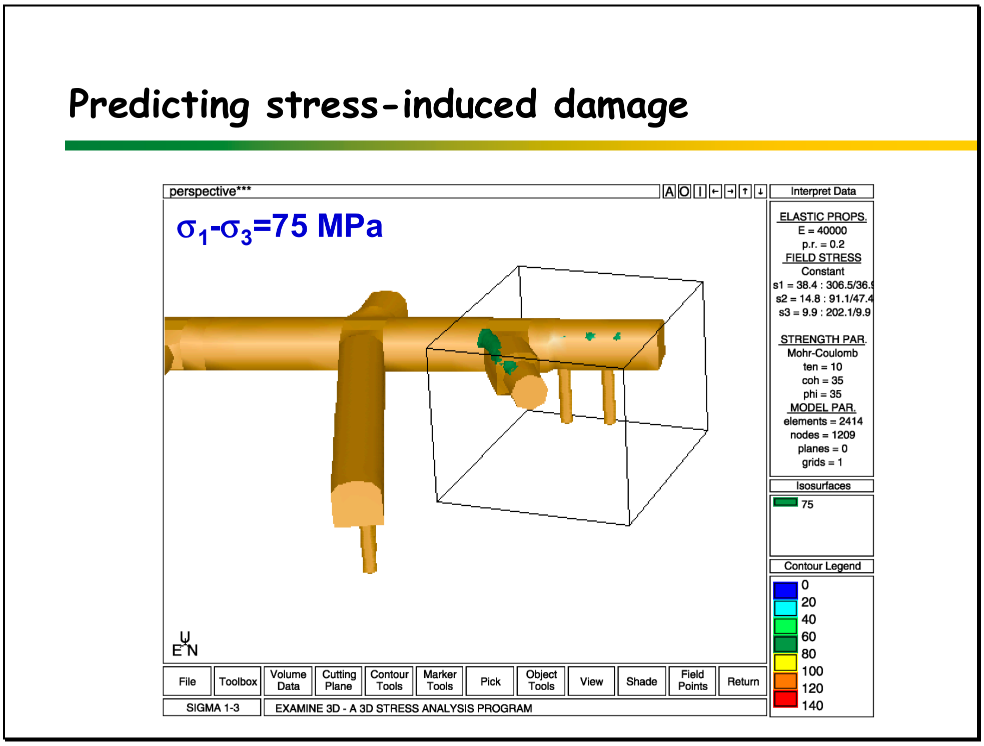


Figure 22

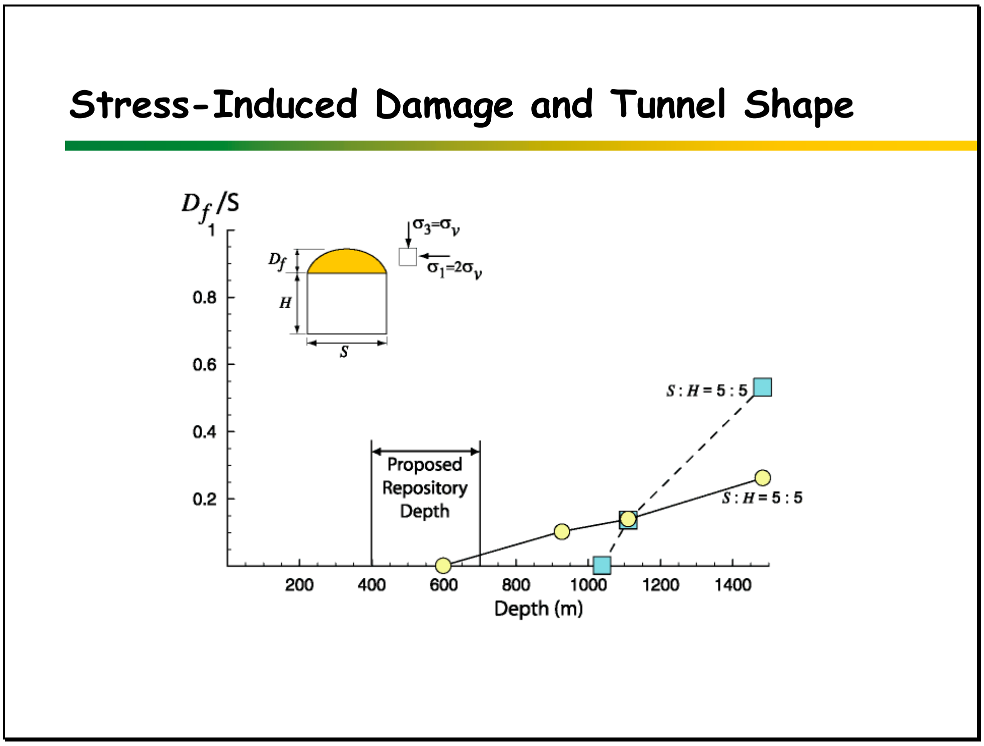


Figure 23

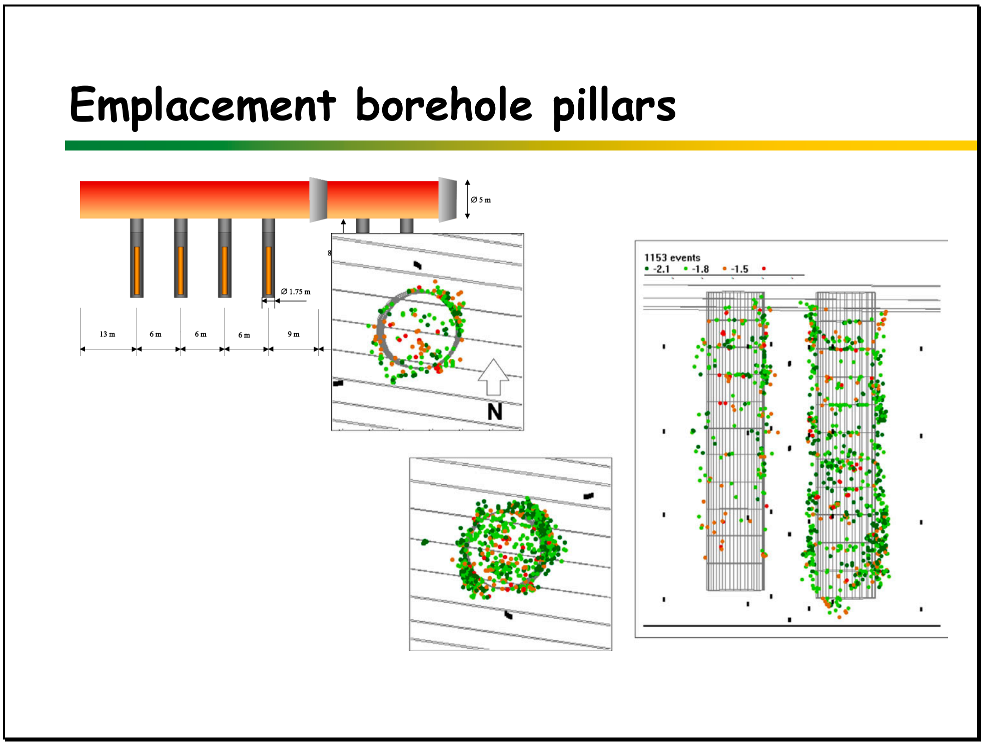


Figure 24

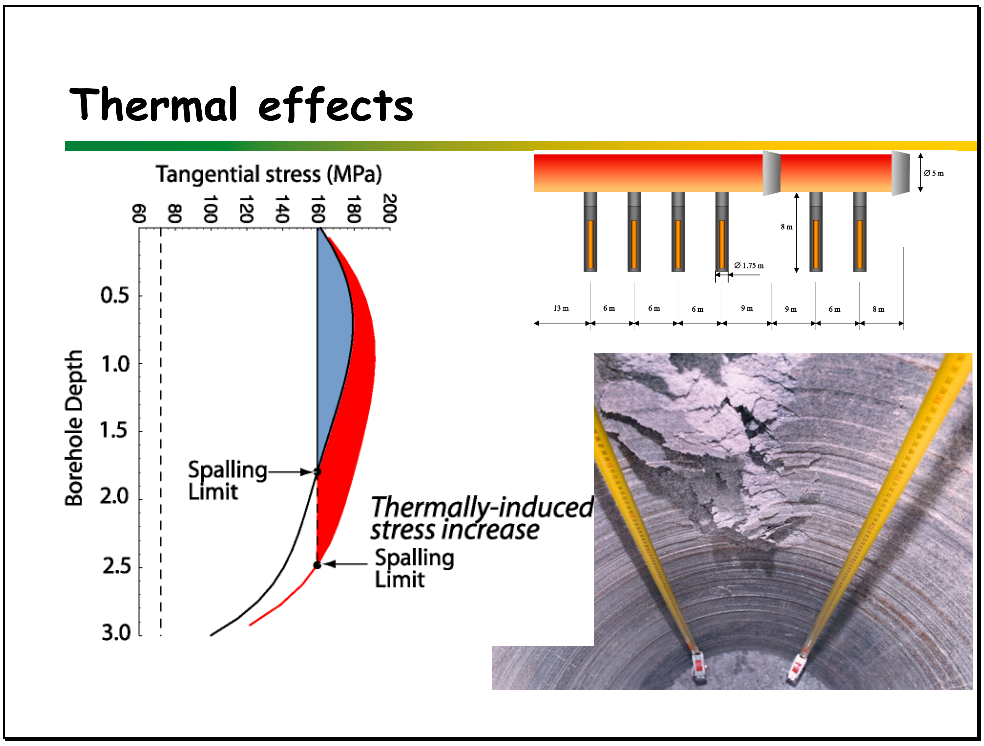


Figure 25

Observations at Underground mines

- Backfill is essential to mining to control stability.
- Types of backfill
  - Sandfill (plain or with cement/flyash)
  - Rockfill (plain or with cement)
  - Paste (cement/flyash)
- Sandfill and pastefill are placed by gravity.
- Rockfill requires waste passes
- Tight placement is often an issue

Figure 26

Typical Mine setup for Sandfill/Pastefill

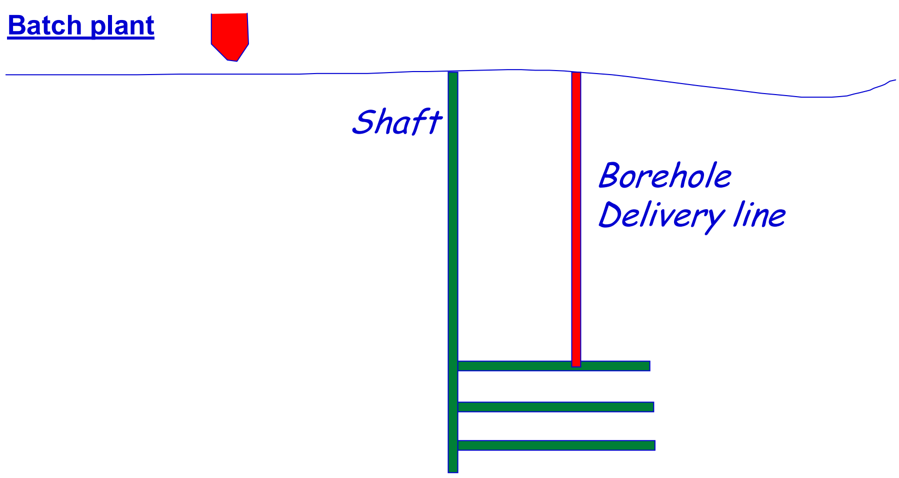


Figure 27

Stope filling

Costs: 6 –10 $/tonne

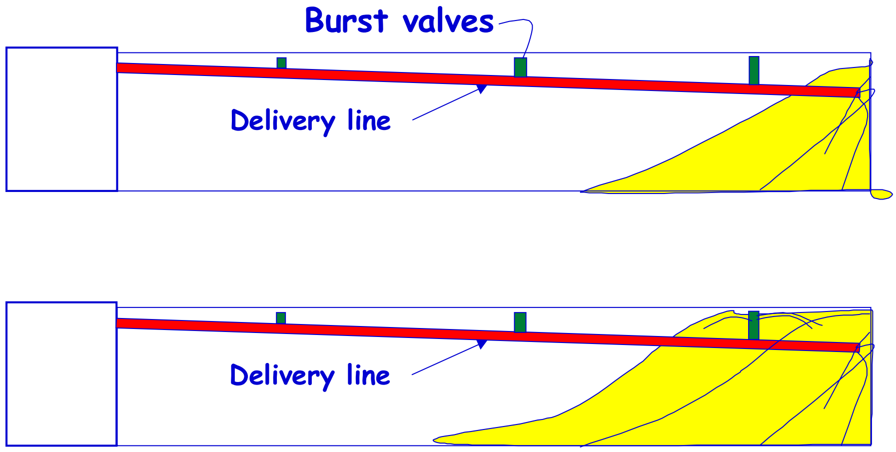


Figure 28

Summary

- Methods used to assess the stability of underground openings must consider the stress path.
- Analysis must consider:
  - Wedge stability
  - Structure/Stress interaction.
  - Brittle failure
- Tunnel shape can be used to improve stability.
- Induced stresses in the pillar between Emplacement boreholes is a function of in situ stress state and spacing between holes.
- Traditional approaches are not appropriate for the analysis of brittle failure.
- The effect of Backfill to minimize the potential for tunnel instability must be quantified.

Paper 8

PRELIMINARY RESULTS FROM 3DEC MODELLING OF A DEPOSITION TUNNEL IN A KBS-3 TYPE REPOSITORY

Rune Glamheden, Sycon Teknikkonsult
Harald Hökmark, ClayTechnology
Rolf Christiansson, SKB

Preface

This paper has been slightly modified since it was presented at the workshop on backfill requirements for a KBS-3-type repository, held on August 27-28 2001 in Äspö HRL. A sub-case, with a gradually stepwise increase in shear strength around the tunnel, has been included. The discussion section has also been considerably expanded.

Background

The tunnel backfill has three functions; 1) to restrict axial flow within the tunnel; 2) to support the tunnel periphery and; 3) to keep the bentonite buffer confined in the deposition holes. The backfill's rock support function has not yet been systematically addressed, and no requirements on the backfill material have been specified from the support point of view. It is, for instance, not clear if the support function requires a backfill swelling pressure, i.e. if a bentonite component in the backfill is necessary. If it can be shown that a substantial swelling pressure is not required, then it might be possible to reduce the bentonite fraction, which will lead to a considerable reduction in cost.

The support function means that deformations leading to a general convergence of the tunnel periphery, or to single blocks being detached from the periphery, are restricted. Rock blocks falling out, as a direct response to excavation, or to drying-induced changes in fracture mineralogy when the tunnels are still open, is a short-term problem during construction and can be dealt with by rock bolting. The backfill support function is a long-term issue and relates to time-dependent deformations in the rock, i.e. rock creep, which is a topic that is conceptually poorly understood [1]. The driving force behind creep is the occurrence of shear stresses, i.e. as long as shear stresses exist, creep may theoretically be possible. At present, we do not know to what extent creep movements could deform the tunnel periphery or which mechanical properties the backfill should have in order to restrict the effects of creep.

No rheological models, i.e. a material models that explicitly take time-dependent stress/strain laws into account, have proved applicable to hard rock masses, intact rock, or rock fractures. It is, however, reasonable to assume that all significant deformations take place along existing discontinuities. Upper bound estimates of deformations, obtained after a very long period of time, may be based on the assumption that fracture movements will continue until the shear loads on all near field fractures have disappeared. Such estimates might be obtained by use of numerical models.

Objectives

The ongoing work presented in this paper is related specifically to the support function of the backfill. The objectives of the project are twofold:

- to arrive at upper bound estimates of convergence in the deposition tunnel that occur due to time-dependent deformation of joints of the tunnel's near field.
- to investigate how the convergence of the deposition tunnel is affected by the use of backfill with different compositions.

The results will hopefully be of help in arriving at a cost-efficient backfill composition in the deposition tunnels.

Strategy

The strategy adopted in the project is to analyse the backfill support function by use of three-dimensional numerical models. The principal idea is to recognise that all significant deformations in rock masses take place along existing fractures and, consequently, that shear loads on fractures is a necessary condition for creep deformations in rock masses. For this reason, a simulation tool that allows for explicit representation of individual fractures, such as 3DEC, has been selected. At present no viscoelastic or viscoplastic fracture material models are available in 3DEC, which means that time-dependence cannot be considered. However, bounding estimates of creep effects can be made by use of the standard built-in elasto-plastic models. By reducing the shear strength of the fractures, the fracture shear loads decrease, and eventually, if the fracture shear strength is set to zero, no further deformations can take place. The accumulated displacements may then represent the maximum possible effects of creep that have taken place over an unknown but long period of time. The technique of stepwise strength parameter reduction is well adapted to codes that employ explicit time-step solution schemes and is an appropriate and often used way of performing sensitivity analyses with such codes, for instance 3DEC. In the first phase of this project, this technique is used to set limit boundaries to creep-induced convergence of a backfilled tunnel.

In the first phase, which is presented here, analyses with idealised fracture patterns and simplified backfill properties are performed. The purpose at this stage of the project is mainly to verify that the chosen calculation tool is suitable and that the problem of mechanical rock/backfill interaction is addressed in a relevant way.

In subsequent phases, more realistic and less idealised fracture patterns and backfill properties will be assumed. In the long term, it may turn out to be necessary to improve the general understanding of rock creep and find ways to perform real time-dependent analyses with inclusion of viscoelastic or viscoplastic fracture constitutive relations [2].

Method

The numerical calculation is performed with 3DEC, a three-dimensional numerical program based on the distinct element method [3]. The program uses an explicit time-step method to solve the equations of motion directly. 3DEC can simulate the response of a jointed rock mass that is subjected to either static or dynamic loading, and allows for large displacements along discontinuities and rotations of blocks.

The rock mass is described as an assembly of rigid or deformable blocks, which interact through deformable contacts. Deformable blocks are subdivided into a mesh of finite difference elements, and each element responds according to a prescribed linear or non-linear stress-strain law. There are several built-in models for deformable blocks, for example isotropic elastic and shear yielding models. 3DEC can consequently be used to simulate intact rock as well as aggregate material.

The discontinuities are treated as boundary conditions between blocks. Relationships governing the movement in both the normal and shear directions determine the relative motion in the joints. The basic model for representing joint material behaviour is an idealised elasto-plastic model with a Coulomb slip criterion. 3DEC also contains a powerful built-in programming language, *FISH*, that enables the user to define new variables and functions and to develop new material models.

An important aspect when solving numerical problems with 3DEC is to decide when the model has reached equilibrium or if it will reach equilibrium at all. There are several features built into 3DEC to assist in making this decision. Normally the equilibrium is evaluated by use of different types of time-record diagrams of selected variables. For the actual problem time-record diagrams of velocities, deformations and stresses in the model are used.

Model description

The models used are on the tunnel scale with dimensions sufficient to accommodate a representative fracture system. The box-shaped models include a horseshoe-shaped deposition tunnel according to SKB layout E. Deposition holes are not included. Stress magnitudes and stress anisotropy are in accordance with conditions prevailing in the prototype area in Äspö HRL. The stress tensor orientation is idealised with principal stresses coinciding with the symmetry planes of the excavation geometry.

Model geometry

To reduce the model size, the deposition tunnel is modelled with a plane of symmetry (mirror-plane) perpendicular to the tunnel axis. The tunnel length studied in the model is only 10 m. The width and height of the tunnel are 5.5 m and 5.5 m, respectively. The width and height of the mesh are 33 m and 33 m, respectively, see Figure 1.

The joint system is generalised, assuming three joint sets: a vertical joint set with a strike perpendicular to the tunnel axis, and two joint sets, both dipping at 60 degrees, with strikes parallel to the tunnel axis. The joint spacing is 2.5 m for the vertical joint set and 1.15 m for the two other sets. This gives a fracture density of approximately 2 units of fracture area per unit of rock volume, while the rock mass surrounding the TBM tunnel in Äspö HRL has a density of 0.5 m^{-1} [4]. Joints are only generated over half the tunnel length within a region of 16.5 m by 16.5 m. A number of construction joints are added to facilitate mesh generation. These joints are locked and do not influence the calculations.

The directions of the coordinate axes are also shown in Figure 1. The x-axis is horizontal, the y-axis vertical and the z-axis parallel to the tunnel axis.

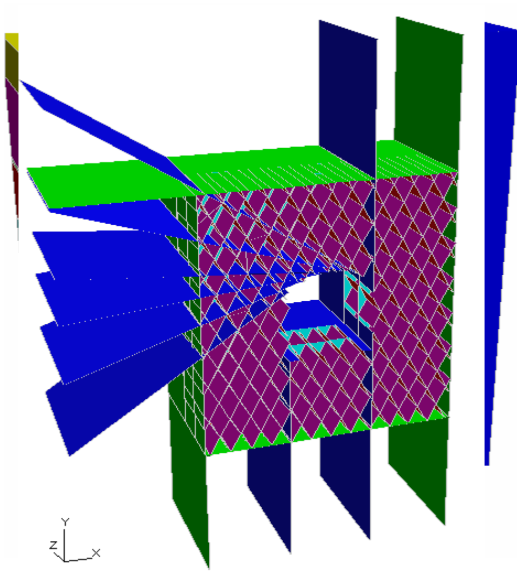


Figure 1 *Geometry of the model. The figure on the left shows the block geometry of the model and the figure on the right shows the joint configuration. The joints with longer extension are joints for mesh generation, which do not influence the calculations.*

In situ stress conditions

The in situ stress conditions adopted for the modelling are, as regards magnitudes, in accordance with those prevailing in the prototype area in Äspö HRL at 450 m depth, see Table 1. The stress tensor orientation is idealised, with the major principal stress being horizontal and perpendicular to the tunnel axis. Within the limits of the model, the horizontal in situ stress field can safely be assumed to be constant with depth.

Table 1 In situ stress conditions evaluated at 450 m depth in the prototype area in Äspö HRL.

Parameter	Symbol	Magnitude (MPa)	Strike
Major horizontal in situ stress	σ_H	32	x-axis
Minor horizontal in situ stress	σ_h	10	z-axis
Vertical in situ stress	σ_v	ρg(depth)	y-axis

Boundary conditions

All boundaries are fixed in the normal direction. This results in a small underestimate of displacements caused by disturbances in the interior of the model. The boundaries are, however, sufficiently far away from the region where disturbances take place to warrant that this boundary effect can be neglected.

Material properties

The selected material properties for the rock mass and the backfill are listed in Table 2. The blocks are assigned the properties of an elastic isotropic material. The joint behaviour is linear-elastic in the normal direction and elasto-plastic with Mohr-Coulomb slip failure in the shear direction.

The backfill is assigned elasto-plastic material properties. The elastic properties are obtained from oedometer modulus determinations performed on TBM muck from the Prototype Tunnel in Äspö HRL [5]. The stress interval in these tests was 0-1.5 MPa. The yield function used in the model is a Mohr-Coulomb failure criterion with tension cut-off and no cohesion. The properties are listed in Table 3.

Table 2 Initial rock mass properties.

Parameter	Symbol	Value	Unit
Young's modulus	E	40	GPa
Poisson's ratio	υ	0.2	
Density	ρ	2700	kg/m^3
Joint normal stiffness	k_n	100	GPa/m
Joint shear stiffness	k_s	50	GPa/m
Joint cohesion	c_j	1	MPa
Joint tensile strength	σ_{jt}	0	MPa
Joint friction angle	ϕ_j	35	deg
Joint dilation angle	i	5	deg

Table 3 Properties of backfill.

Parameter	Symbol	Value	Unit
Bulk modulus	K	15.5	MPa
Shear modulus	G	7.1	MPa
Cohesion	c	0	MPa
Tensile strength	σ_t	0	MPa
Friction angle	ϕ	40	deg
Dilation angle	ψ	10	deg

Calculation cases

The modelling work is divided into three calculation cases:

Case 1

The deposition tunnel is modelled without consideration given to any backfill or rock reinforcement. The aim is to get an estimate of the upper limit of the convergence that could develop due to rock mass creep.

Case 2

The deposition tunnel is modelled only taking account of the effect of the backfill swelling pressure. A constant pressure of 0.1 MPa is applied on the tunnel circumference. The result will give an indication of the contribution of the bentonite component to the overall backfill support function.

Case 3

The deposition tunnel is modelled considering the backfill as a friction material with specified bulk modulus and shear modulus but without any swelling pressure. The results from this simulation will give an idea of the relationship between tunnel convergence and the backfill compressibility.

Case 1 and 3 are also analysed for a sub-case regarding the mode of strength reduction in the assumed creep zone around the tunnel. In case 1a and 3a the shear strength is reduced stepwise with a constant value in the entire creep zone. In case 1b and 3b the shear strength is changed stepwise but with a gradually increasing value from the tunnel periphery, see Figure 2.

Modelling sequence

The cases described above all include the following calculation steps: (a) setting of initial, pre-mining equilibrium conditions, (b) excavation of the tunnel and (c-j) stepwise reduction of shear strength in the rock mass by changing the cohesion and friction angle in the joints. The friction angle values are presented for each calculation step in Figure 2. The cohesion is reduced to zero in step (c). The displacements are reset after step (a) and step (b). In this way it is possible to separate the direct effects of excavation from effects of creep.

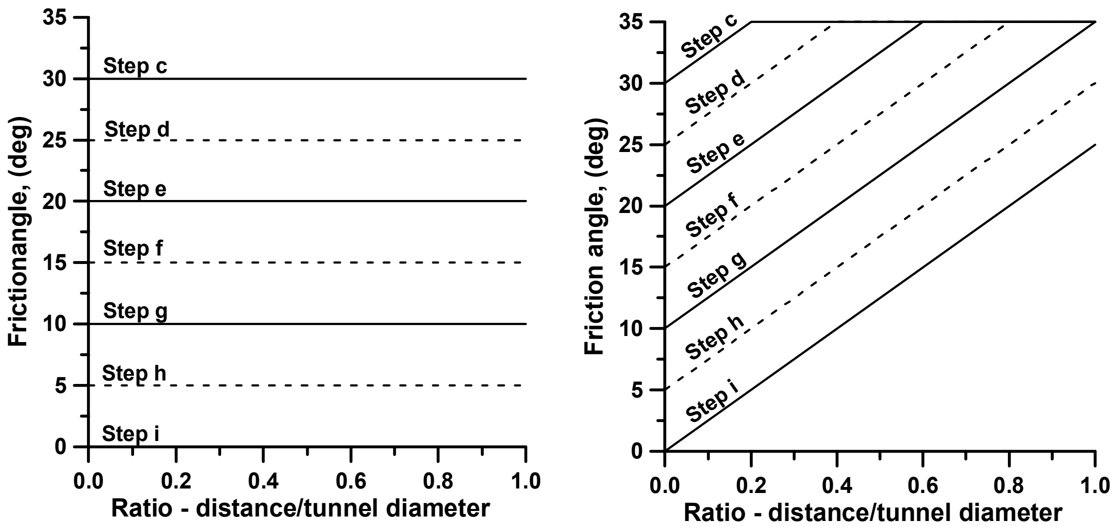


Figure 2 *Modelling steps in the calculation sequence.*

Results

In this section some examples of the results are given. The results are only preliminary and have not been analysed in detail yet, since the modelling is still ongoing.

Tunnel displacements

The displacement field predicted by the numerical model during the tunnel excavation and the phase of reduced shear strength in the rock mass are presented in Figure 3. The displacements caused by the excavation are largest in the tunnel wall while those caused by shear strength reductions, i.e. by creep, are largest in the tunnel floor. At low friction angles, a distinct wedge movement is developed in the tunnel roof and the tunnel floor.

Tunnel excavation.

Max displacement 6.5 mm.

Strength reduction. $\phi = 10$ deg.

Max displacement 28.6 mm.

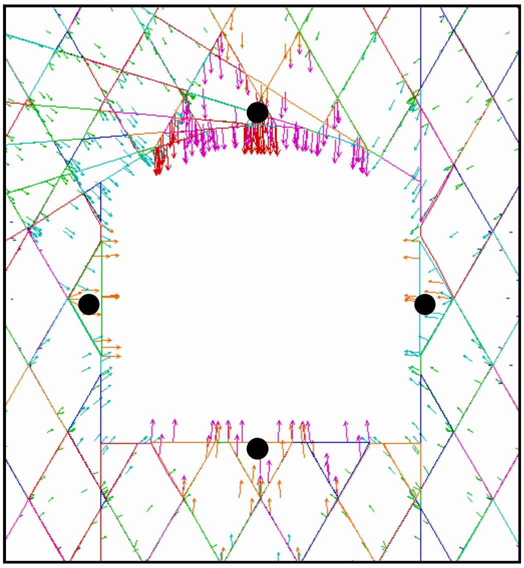


Figure 3 *Calculated displacement vectors in a vertical plane of the deposition tunnel for Case 1a. The displacements are set to zero between excavation and strength reduction. Circles in the right part indicate approximate location of time-record points.*

In Figure 4 the displacement during shear strength reduction in the rock mass versus friction angle for the three different calculation cases, including sub-cases are presented. In case 1a and 1b, equilibrium is never reached for the zero friction angle. Case 2 theoretically should reach equilibrium, since the weight of the loose rock blocks is less than the applied pressure on the tunnel periphery. However, the values given in the graph at zero friction angle, are not readings at equilibrium but readings made when the calculation had to be interrupted because of lengthy run times.

The calculated displacements are of similar magnitudes in respective sub-cases, (a) and (b), at friction angles above ten degrees. It is only after a reduction to five degrees friction angle that the results diverge.

As expected, the smallest displacement is calculated for case 3. This model reaches equilibrium even at a zero degree friction angle. The calculated displacement is almost equal in the tunnel roof and tunnel floor with a value of 115 mm in case 3a and 30 mm in case 3b. The displacement in the tunnel walls amounts to approximately 100 mm in case 3a and 20 mm in case 3b. The specific assumption in case 3b, i.e. a gradual increase in shear strength around the tunnel, obviously reduced the calculated final displacement significantly.

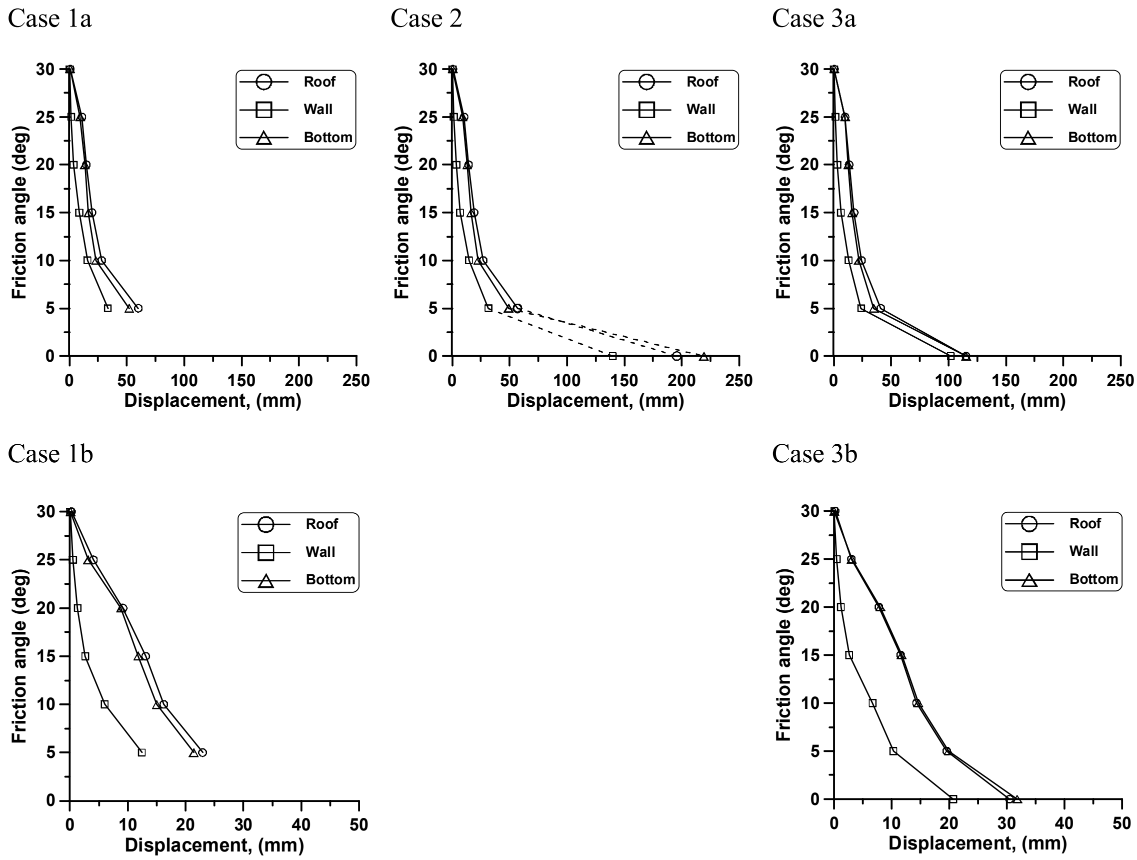


Figure 4 *Displacement versus friction angle from time-record points on the tunnel periphery marked in figure 3. Because of symmetry, only one wall point is included.*

Stresses in the rock mass

In Figure 5 the stresses in the rock mass after excavation and shear strength reduction are shown. The stress field after excavation bears a resemblance to a stress field calculated under elastic conditions, while after shear strength reductions almost stress-free wedges have formed in the roof and floor regions.

Tunnel excavation

Max compressive stress 119.1 MPa.

Strength reduction. $\phi = 10$ deg.

Max compressive stress 78.4 MPa.

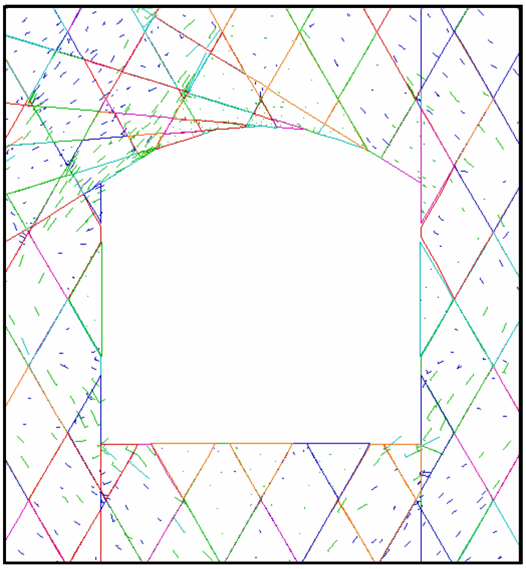


Figure 5 *Calculated stress field in a vertical plane of the deposition tunnel for Case 1a .*

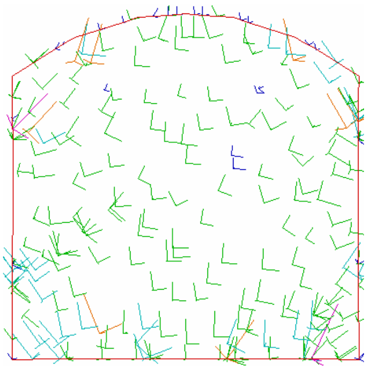
Stresses in the backfill

The calculated stresses in the tunnel backfill are presented in Figure 6, as stress tensors and as diagrams of the stresses in the centre of the backfill versus friction angle. In both cases the stresses in the backfill are moderate and almost isotropic at friction angles above 5 degrees. At additional reduction of the friction angle, the stress magnitudes rapidly increase at the same time as the stress state becomes more anisotropic in case 3a, while in case 3b the stresses remain moderate. This response of the model is logical and consistent with the obtained tunnel periphery displacements, see Figure 3.

Case 3a

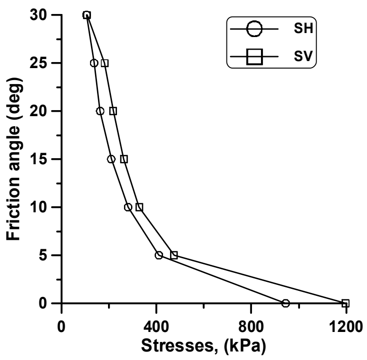
Strength reduction. ϕ = 10 deg.

Max compressive stress 956.2 kPa.



Case 3a

Stresses in vertical and horizontal direction in the centre of the backfill.



Case 3b

Stresses in vertical and horizontal direction in the centre of the backfill

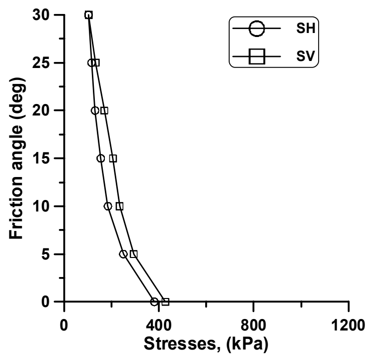


Figure 6 *Calculated stresses in the tunnel backfill during shear strength reduction for case 3a and 3b.*

Discussion

Failure mode

The types of failure that occur in different rock masses under low and high in situ stress levels are described by Hoek [6]. Three main failure mechanisms are identified:

- failure in jointed rock of good quality, subjected to low stresses due to unfavourable orientations of joints
- failure in jointed or massive rock of good quality due to high stresses
- failure of a jointed rock of fair quality subjected to high stresses.

The conditions assigned to this basic model result in a failure that generally agrees with the last failure mode described above. The results are judged not to be in true agreement with the type of failure that could be expected for the rock conditions observed at Äspö HRL.

Fracture geometry

The fracture geometry assumed in the model has most probably predicted an unrealistically large tunnel convergence. The assumed joint orientation, joint density and extension of the joints are all judged to be on the conservative side for hard rock intended for a deposition tunnel. The joint orientation, with the two important joint sets parallel to the tunnel axis, makes the response of the model to a great extent two-dimensional, without locking effects. The joint density assigned to the model is an overestimation by a factor of four compared to the density observed in the TBM tunnel at Äspö [4]. Finally, the joint extent in the model was in the order of 15-20 m, while DFN models of the TBM tunnel rock mass indicate that single fractures on the average have smaller extent. These conservative assumptions ought to be corrected by further modelling in order to represent the conditions at Äspö HRL in a better way.

In situ stress orientation

The in situ stress orientation, with the major stress normal to the tunnel axis, is also an assumption that represents a worst-case [7]. Turning the horizontal in situ stresses 90 degrees in relation to the tunnel axis will result in a more uniform stress field and reduced displacements in the analysed tunnel cross section.

Creep region

Shear loads on fractures is a necessary, but probably not sufficient, condition for creep movements. Here it is assumed that some sort of additional disturbance is necessary to initiate creep. The disturbances caused by the tunnel, i.e. stress redistribution effects and direct excavation damage effects, are usually assumed to reach about one tunnel diameter from the tunnel periphery. In this study, creep is assumed to take place only within this zone. Therefore, fractures were not explicitly modelled outside the 16.5 m by 16.5 m region shown in Figure 1. Given this restriction, the worst-case assumption is that complete shear stress relaxation will occur uniformly within that region. This corresponds to the results shown in the upper part of Figure 4. A more realistic assumption may be that fracture shear stress relaxation is significant close to the boundary where disturbances are large and gradually smaller at larger distances. This corresponds to the results shown in the lower part of Figure 4.

Fracture properties

In this study, a standard elasto-plastic material model has been used to account for the contribution of fractures to the overall behaviour of the near field rock mass. It is important to keep in mind that friction angle and cohesion are model parameters, not fundamental physical properties. The stepwise reduction of the joint friction angle was performed in order to allow for relaxation of fracture shear stresses and does not imply that fracture properties actually change over time, although this may be the case to some extent. The real mechanisms by which shear stresses are relaxed may include continuous redistribution of forces acting on fracture surfaces with local movements occurring as a result of local micro-scale yielding of over-stressed and weak fracture contacts, with accompanying successive transfer of stresses to neighbouring stronger contacts [8]. Also actual alteration of fracture properties may contribute, such as changes in mineral composition, density, and moisture content of fracture infillings. No attempt has been made in this study to quantify or describe any of these mechanisms.

In reality, it is likely that creep is strongly retarded and eventually dies out when the fracture shear/normal stress ratio falls below some threshold value. For intact rock, creep models, based on the assumptions that it takes stress/strength ratios above 1/3 in order to maintain the creep process, have been proposed [8]. If similar relations also hold for rock fractures is not clear, but it is obviously very conservative to assume that creep can continue until all shear stresses have disappeared. Setting the threshold ratio to 1/6, for instance (5 degrees friction angle), would limit the convergence estimate for the a-cases (uniform loss of strength in the entire creep region) to 100 mm, with everything else remaining unchanged (c.f. Figure 4, upper). For the case of a graded loss of strength, the upper bound estimate would be 50 mm (c.f. Figure 4, lower).

Backfill compression modulus

The calculated stress level in the backfill at zero friction angle lies well within the stress range used for measuring the backfill compressibility in the laboratory. Within that interval, no strain-hardening was found. 3DEC has built-in strain hardening material models that could be used to represent the backfill if new experimental data would suggest it was necessary. At present, however, the constant stiffness model seems adequate.

Conclusions

Backfill swelling pressure

A preliminary conclusion that can be drawn from the modelling is that a small swelling pressure, in the order of 0.1 MPa, does not seem to have a visible influence on the tunnel convergence induced by creep in fracture planes. This is supported by the small differences found between case 1 (no tunnel support) and case 2 (constant pressure on the periphery) for friction angles over 5 degrees. However, this study has not addressed questions related to the initiation or development of brittle failures in intact rock caused by high tangential stresses very close to the periphery. To prevent this type of failure, small confining stresses may be effective. Small swelling pressures are also effective for maintaining unbroken contact between rock and backfill, thus preventing axial flow paths forming within the tunnel periphery.

Backfill stiffness

The backfill considered in this study corresponds to crushed rock, compacted to between 85% and 95% of Proctor density. For this type of backfill, the following can be concluded:

- For conditions with complete loss of frictional interaction between blocks, which might have given serious instability in the case of an open tunnel (Fig. 4, left part), the backfill restricted the tunnel convergence to 20-25 cm in case 3a and to 5 cm in case 3b (Fig. 4, right part).

- For conditions found to be stable even without backfill support (friction angle above 5 degrees), the backfill only reduced the tunnel convergence by 20-30%. This indicates that the backfill stiffness must be much higher in order to restrict the time-dependent convergence to a large extent.

Tunnel convergence

Summarising the results presented in section 6 and the points discussed in section 7, and considering all data and model uncertainties, it seems reasonable at this stage to conclude that 100 mm is a conservative upper bound estimate of the tunnel convergence that may occur as a result of creep along fracture planes. Probably 50 mm is a more realistic, but yet conservative, estimate. It seems to be clear that the backfill cannot have any influence on creep-induced convergence, unless the stiffness or the swelling pressure is increased very significantly, perhaps by orders of magnitudes.

For subsequent project steps, one of the objectives should be to increase the confidence in the convergence estimates, or even to get better, narrower estimates. Several ways of accomplishing this should be tried, including literature studies and additional analyses.

Modelling technique

The technique of using shear strength reductions to allow for relaxation of fracture shear loads is a rough and arbitrary way of representing creep in a jointed rock mass without apprehension of the actual time it takes to reach the displacements. However, the results obtained so far indicate that the modelling technique could help to clarify the relative importance of different factors that determine the extent of creep-induced convergence. Some improvements in order to get estimations more close to reality may be worth future study, such as using more realistic joint geometries. It is also desirable to get a better understanding of the factors that control the extension of the creep zone and the mode of strength reduction in the fractures around a deposition tunnel.

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[8] **Pusch, R., Hökmark H., 1993**. Mechanisms and consequences of creep in the nearfield rock of KBS-3 tunnels. SKB TR 93-10, SKB, Stockholm.

Paper 9

Confining stress an fracture growth

Namkak Cho

Figure 1

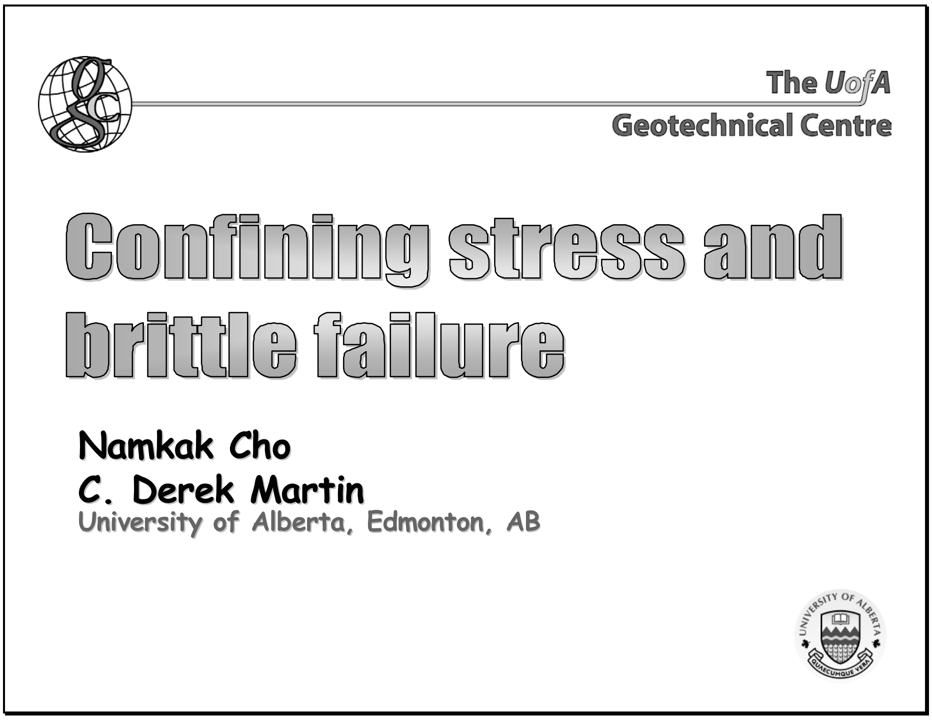


Figure 2

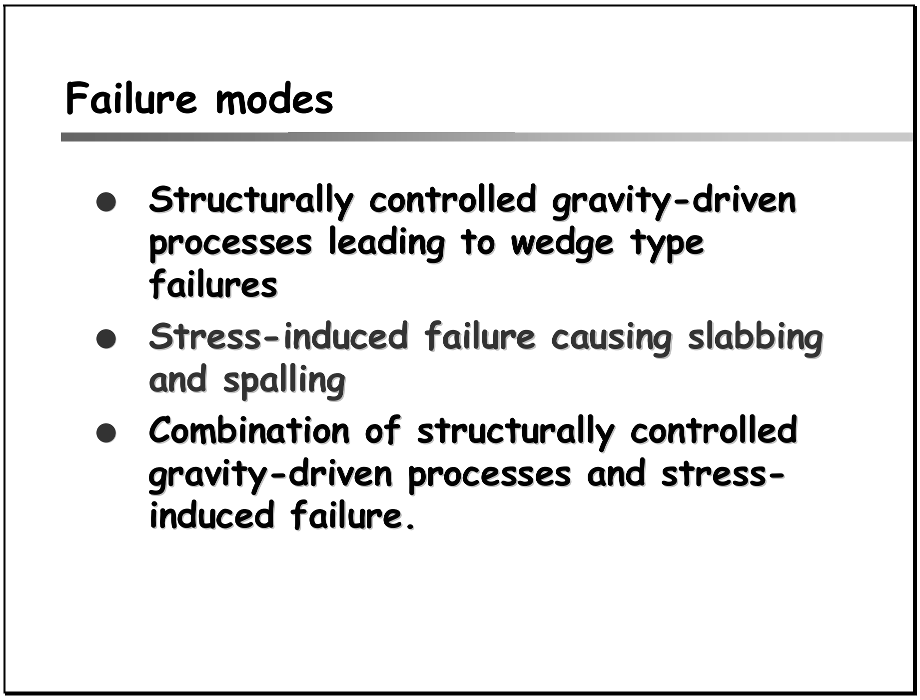


Figure 3

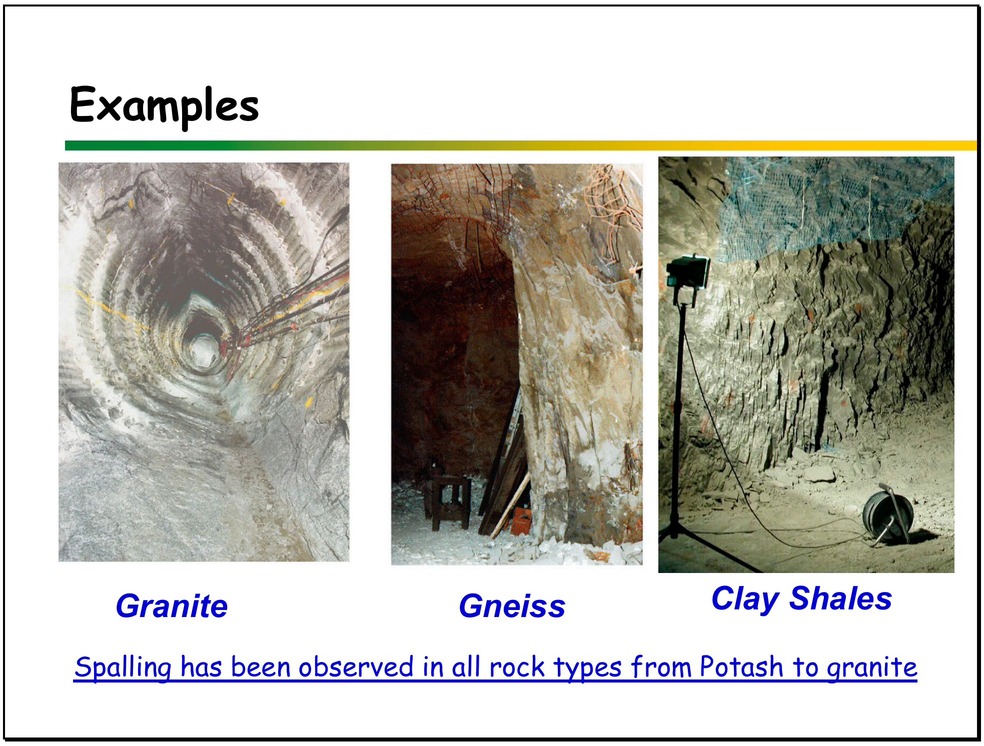


Figure 4

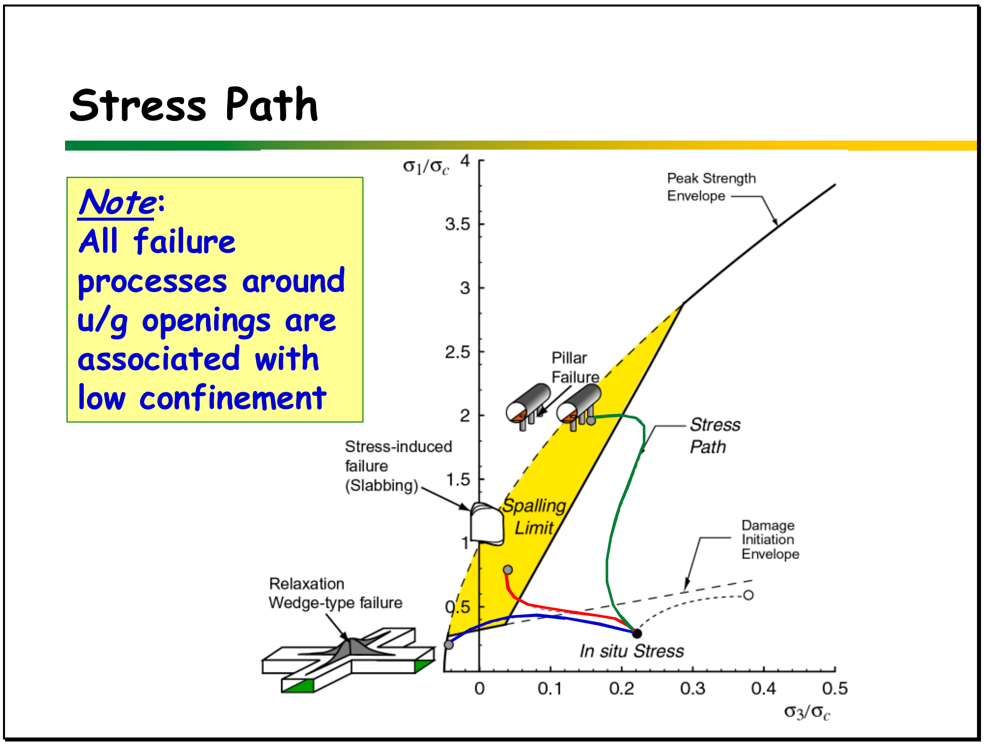


Figure 5

Observations at AECL's URL

- Removal of tunnel muck (~20kPa) from floor of Mine-by test tunnel sufficient to induce slabbing.
- Addition of sand in the TSX experiment bulkhead sufficient to stop all microseismic activity (~10kPa).
- During the heated failure test filling the hole with water in a sealed liner caused the microseismic activity to decrease to below background levels (0.08 AE events/min). When the hole was drained the AE frequency increased to 1.25 events/minute. Maximum pressure at the borehole collar is 100 kPa.

Figure 6

Observations at Underground mines

- Backfill is essential to mining in burst prone ground.
- Backfill usually consists of slurried sandfill either with or without a cement binder. The strength of the backfill with cement is ~ 1MPa.

Figure 7

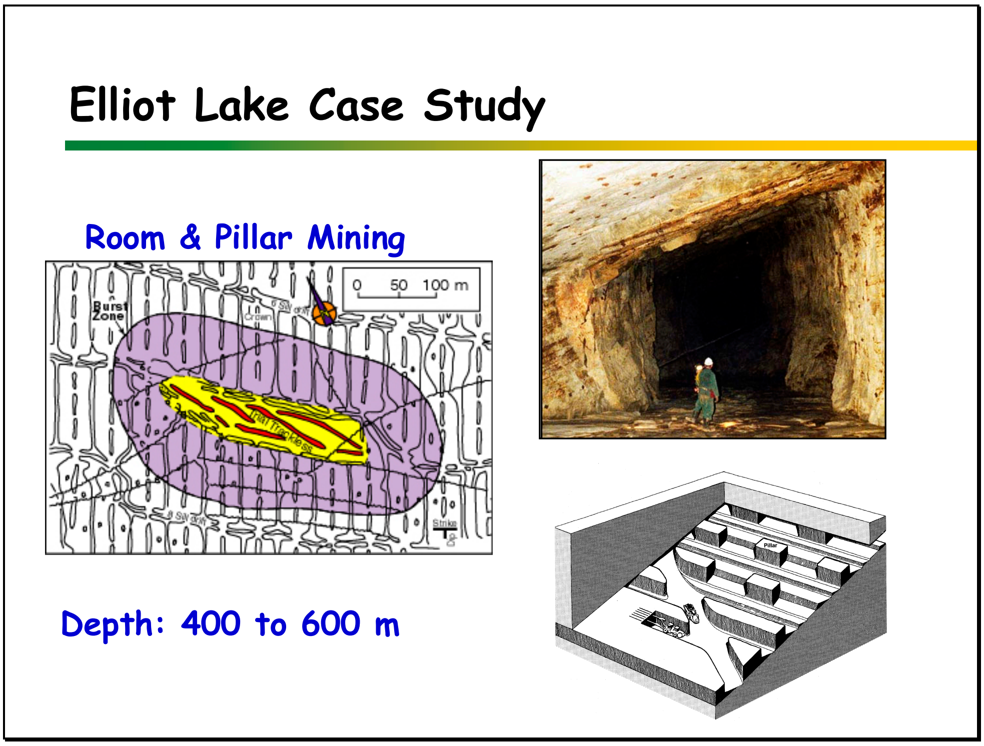


Figure 8

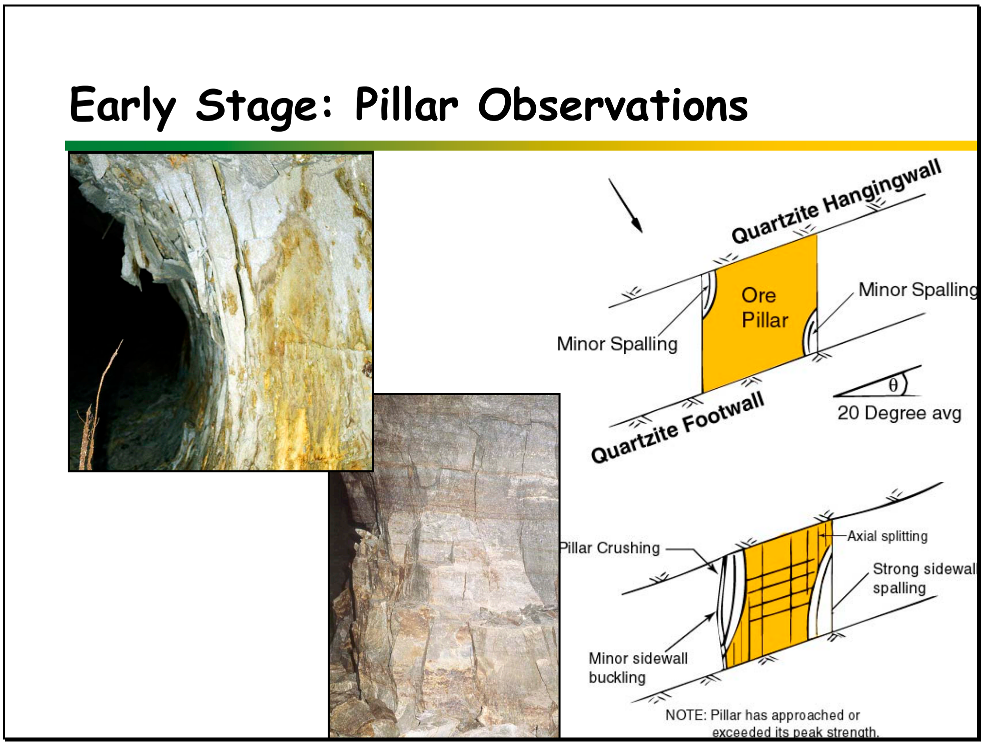


Figure 9

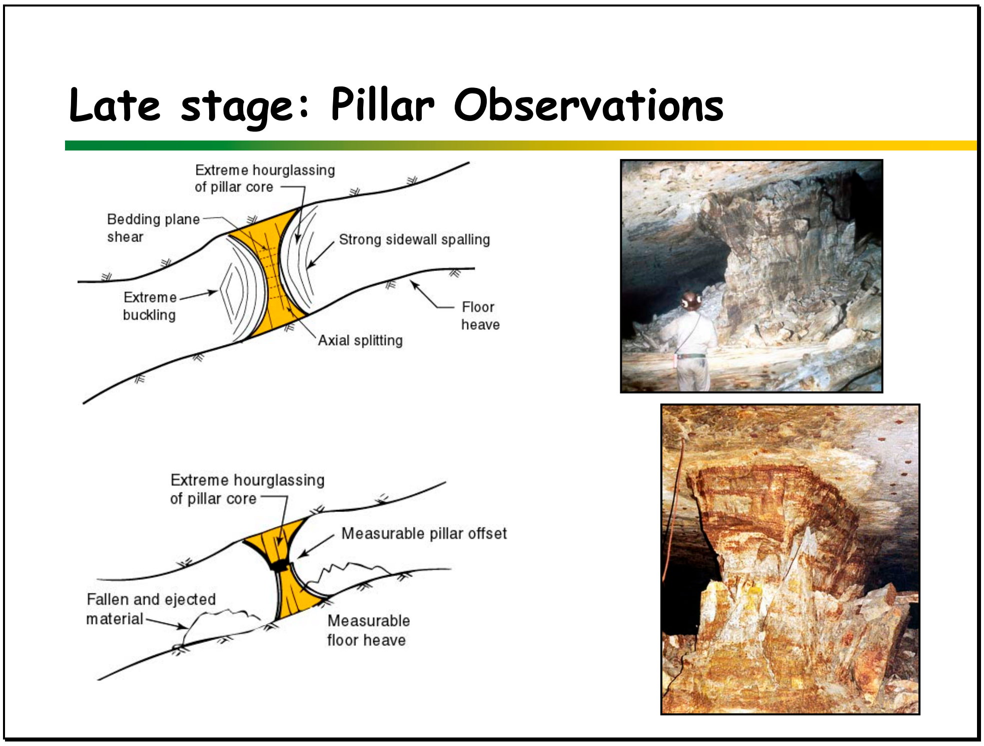


Figure 10

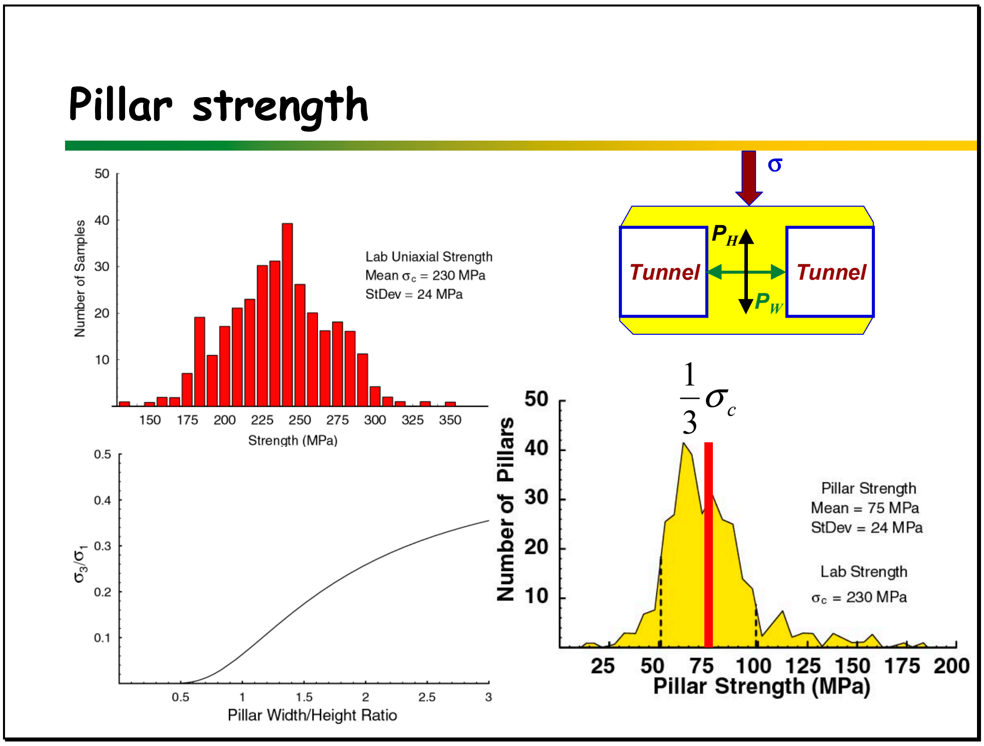


Figure 11

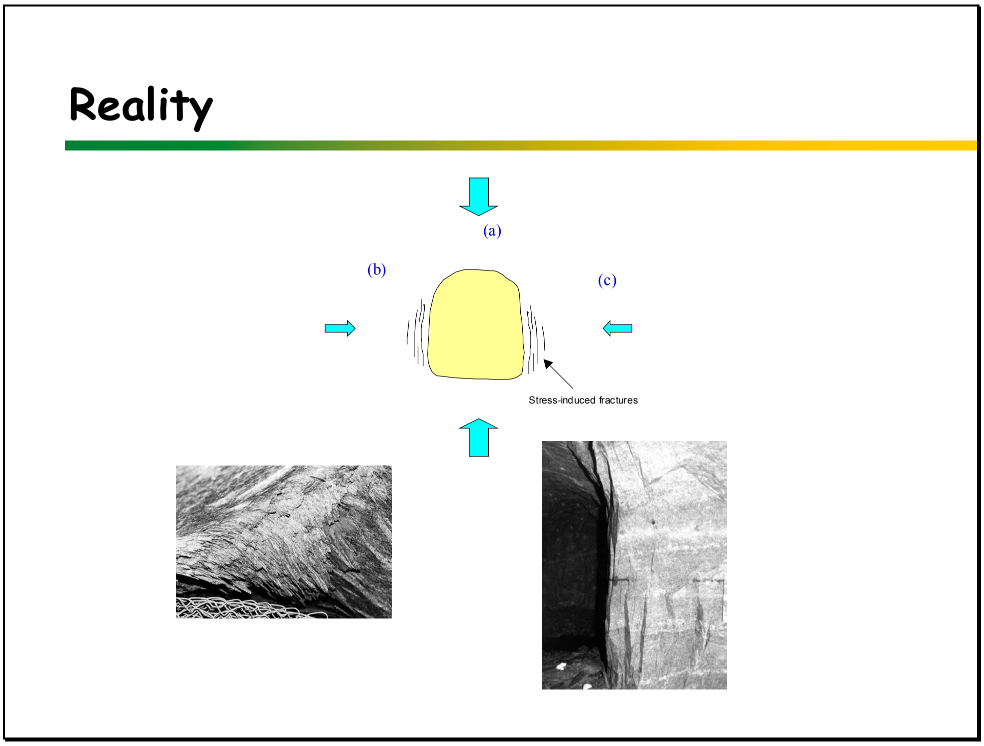


Figure 12

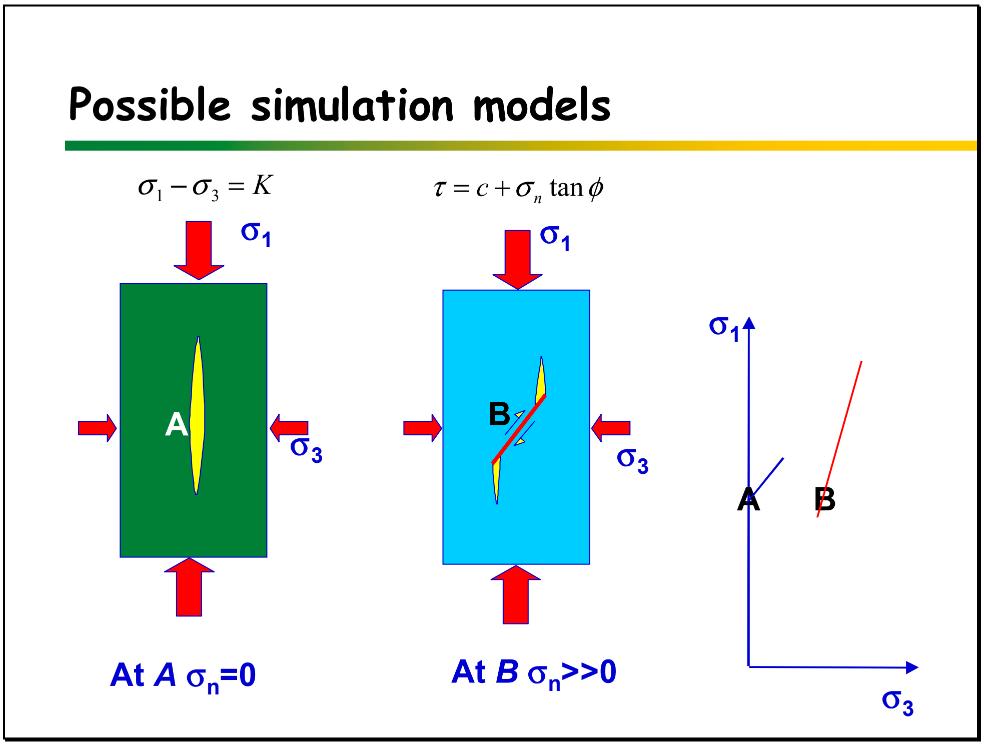


Figure 13

Experimental evidence

after Hoek 1965

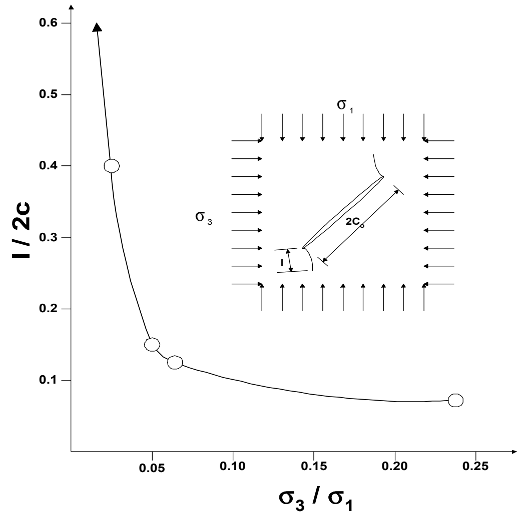
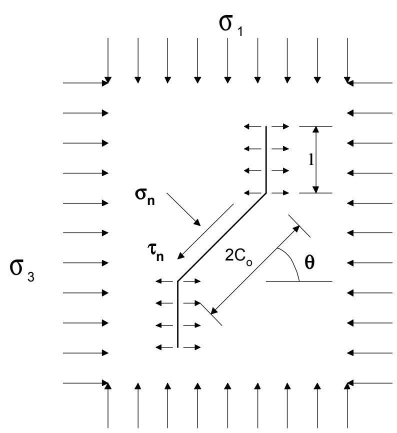


Figure 14

Analytical models (Sliding)

Kemeny and Cook's Model(1987)



$$K_I = \frac{2c_o \tau^* \cos\theta}{\sqrt{\pi l}} - \sigma_3 \sqrt{\pi l}$$

$$\tau^* = \tau_n - \mu \sigma_n$$

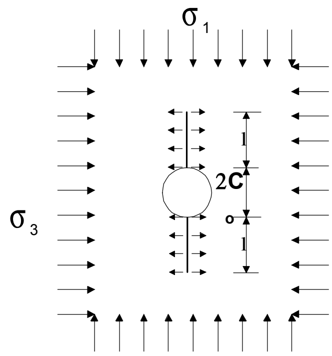
Figure 15

Analytical models (Pores)

Ashby and Sammis(1986)

Cylindrical model

$$K_I = L^{1/2}\left\{\frac{1.1(1-2.1\lambda)}{(1+L)^{3.3}}\right\}\sigma_1\sqrt{\pi c_o}$$



Spherical model

$$K_I = L^{1/2}\left\{\frac{0.62(1-1.8\lambda)}{(1+L)^{4.1}}\right\}\sigma_1\sqrt{\pi c_o}$$

Figure 16

Tensile model

Cai et al, 1999

$$\sigma_3 = \sqrt{\frac{3E(E_o + 2\pi\gamma_s a^2}{8(1-\upsilon^2)a^3}}$$

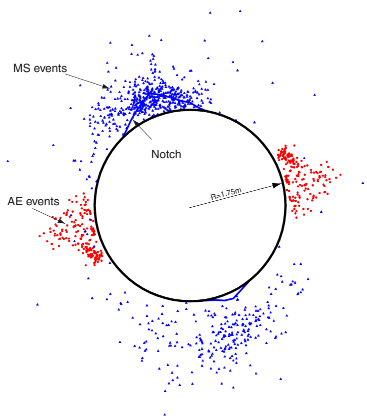


Figure 17

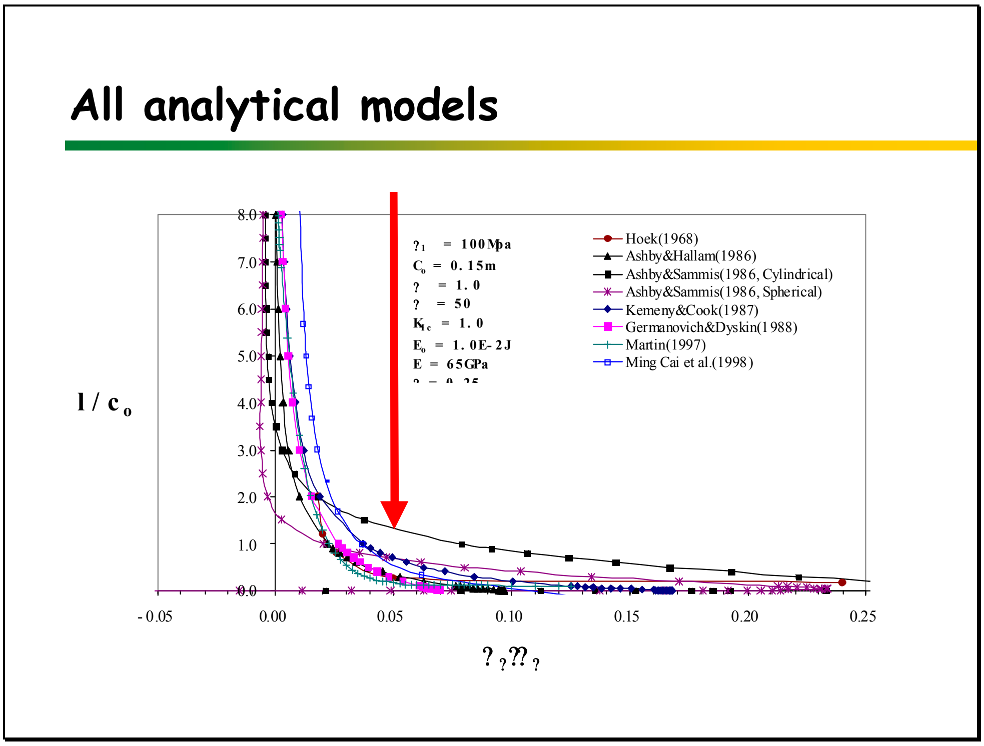
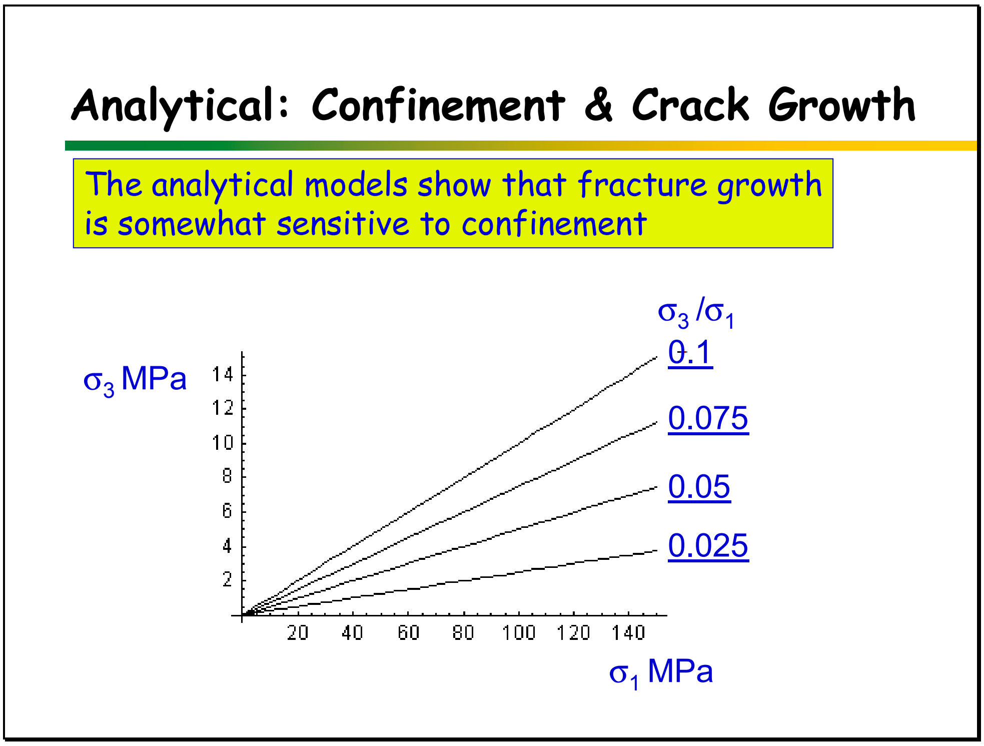


Figure 18



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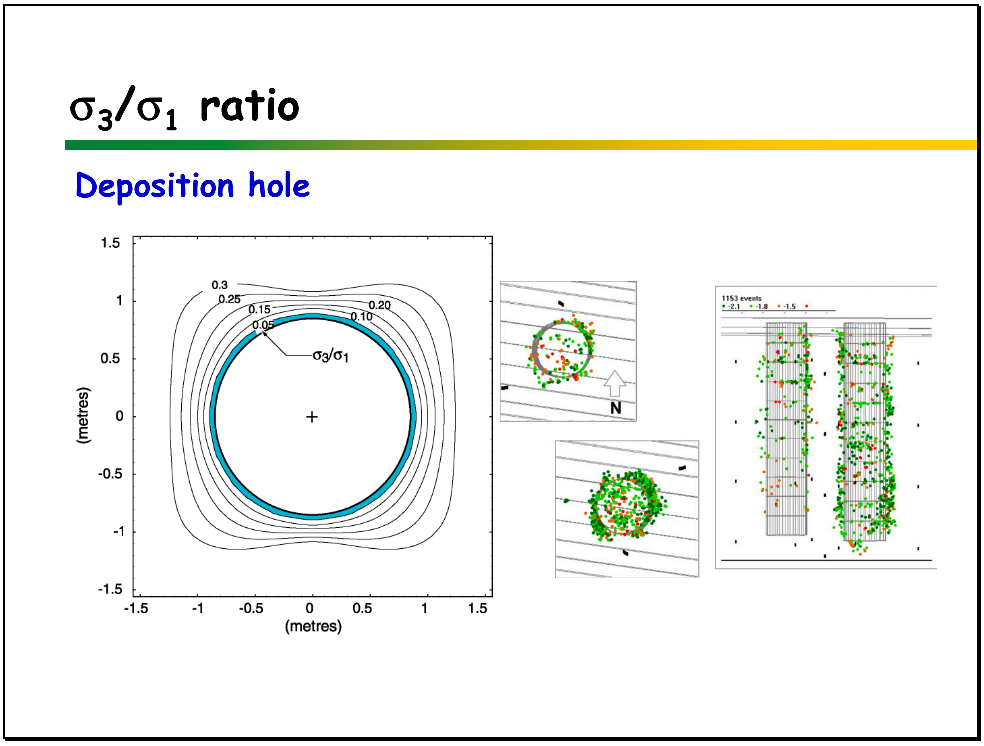


Figure 20

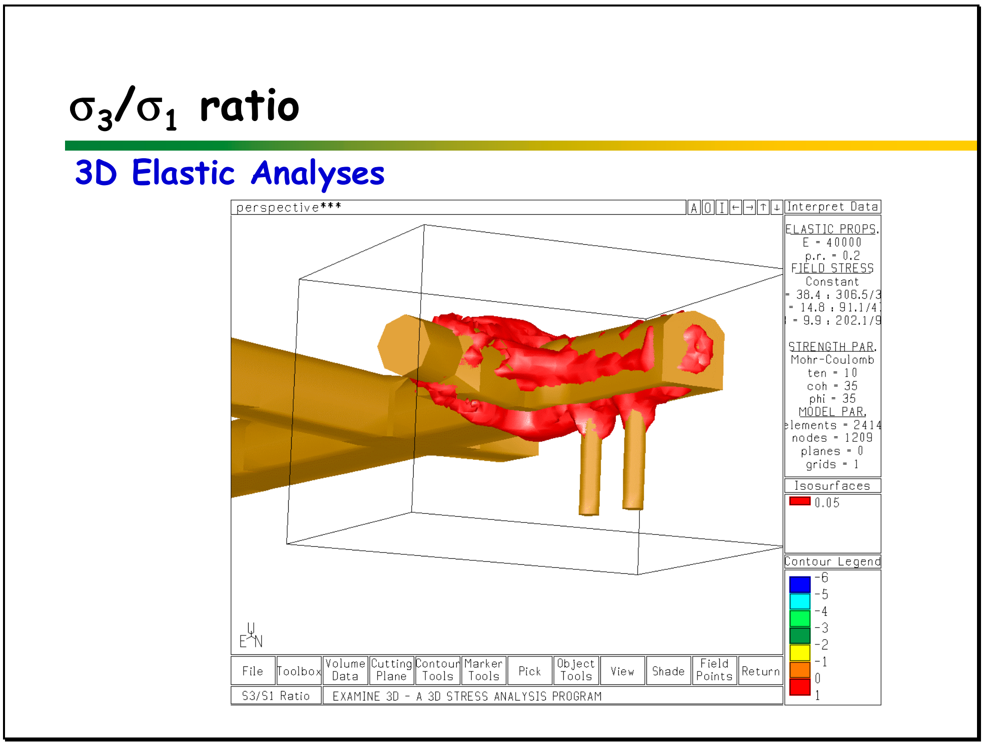


Figure 21

Fracture growth: Numerical methods

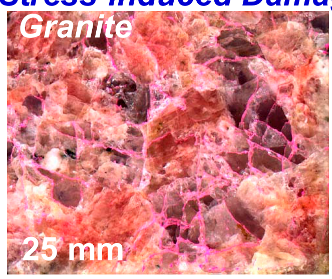
- **Discrete fracture models**
  - DIGS (CSIR, South Africa) Sliding crack
  - Franc2D,3D (Cornell) Finite width crack

- **Discrete Element model**
  - PFC2D,3D

Figure 22

Micro-mechanics

Stress-induced Damage



Thermal Damage

Tunnel Spalling

100 mm

Figure 23

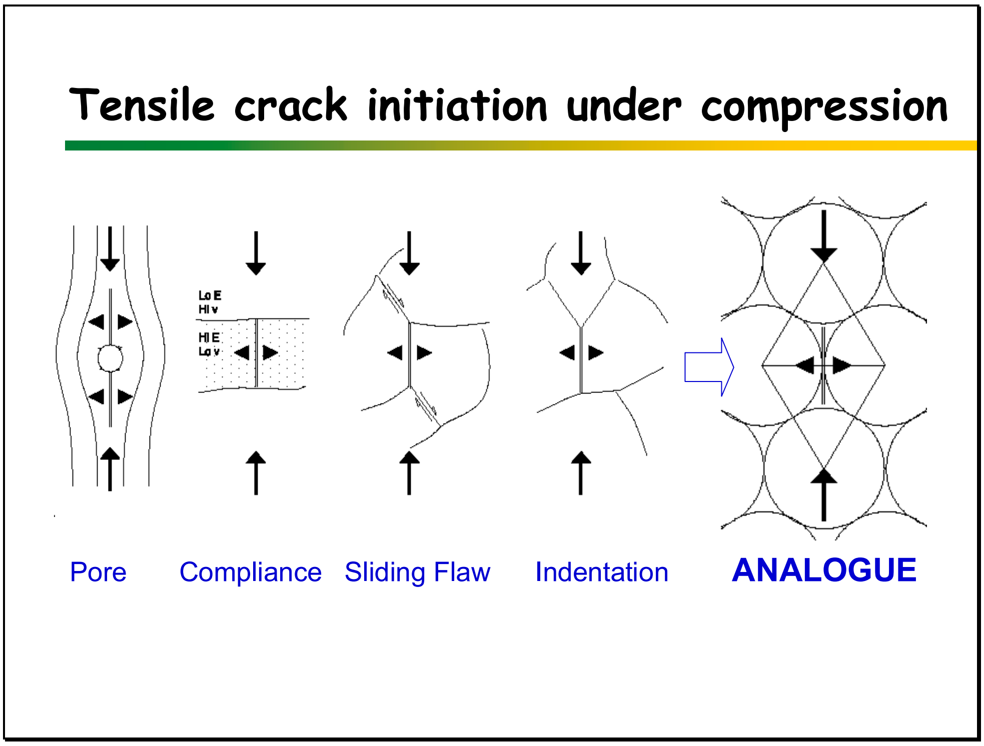


Figure 24

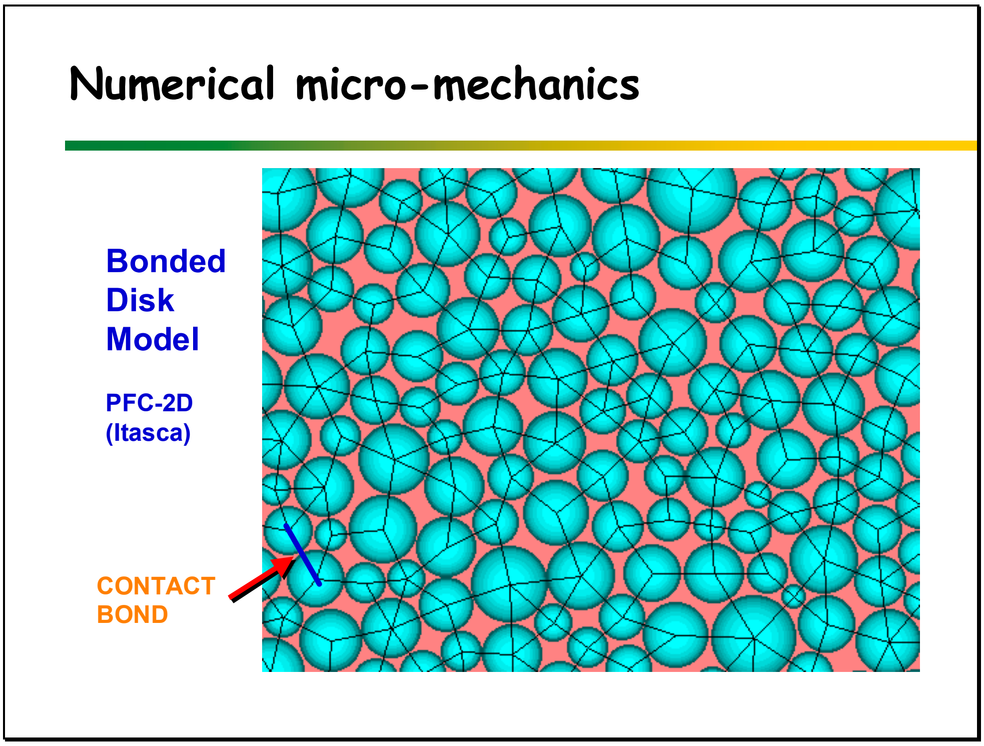


Figure 25

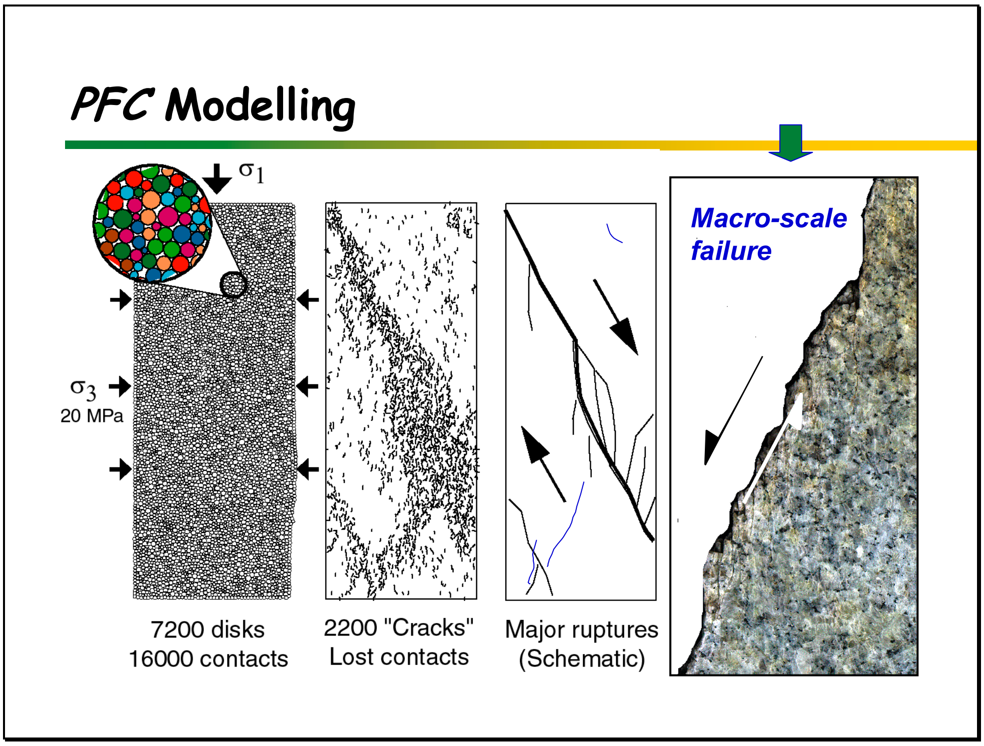


Figure 26

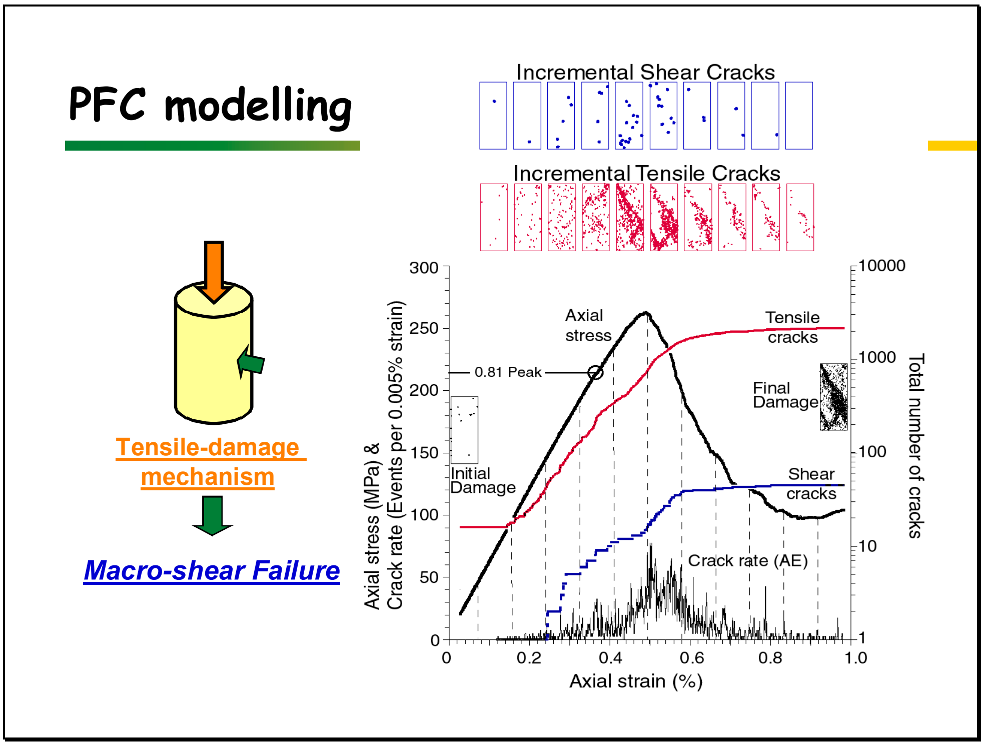


Figure 27

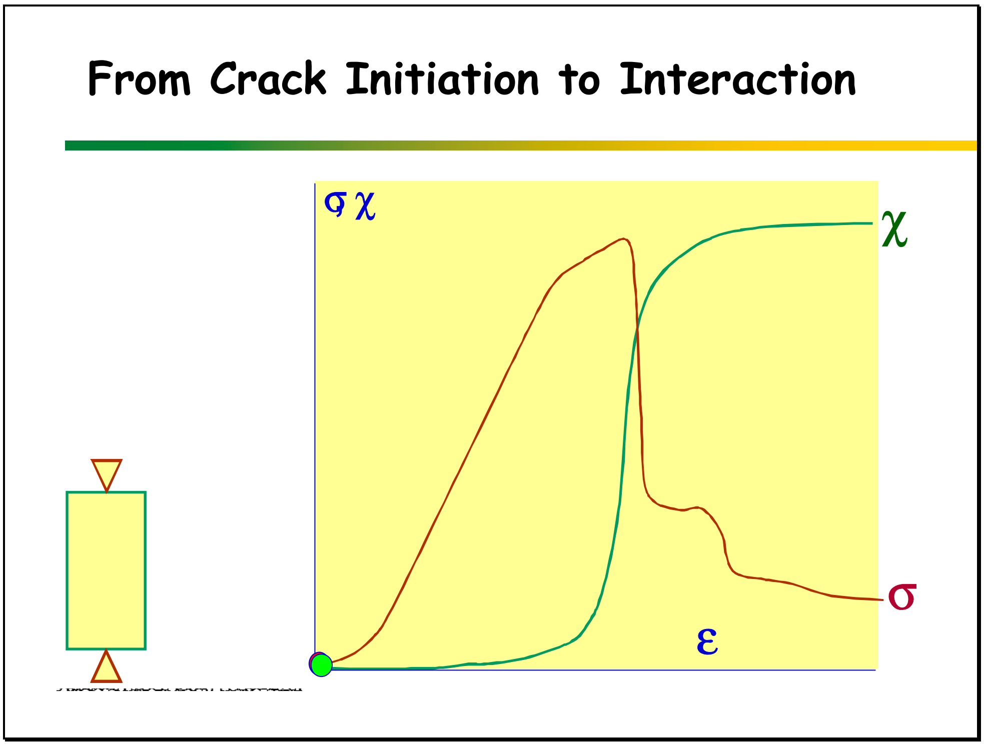


Figure 28

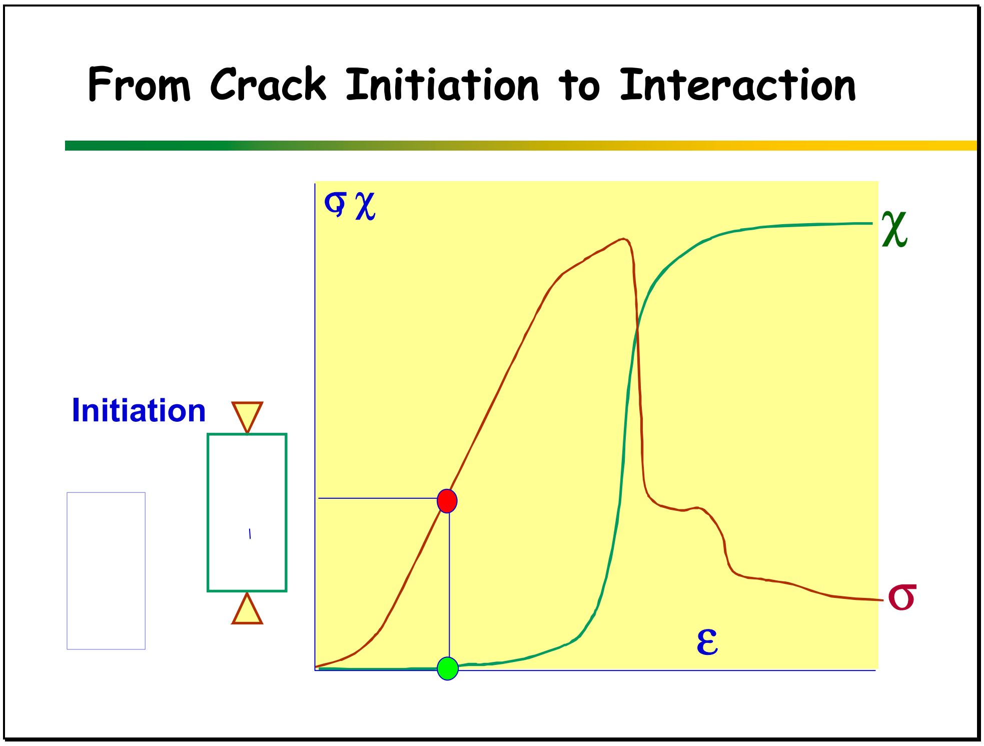


Figure 29

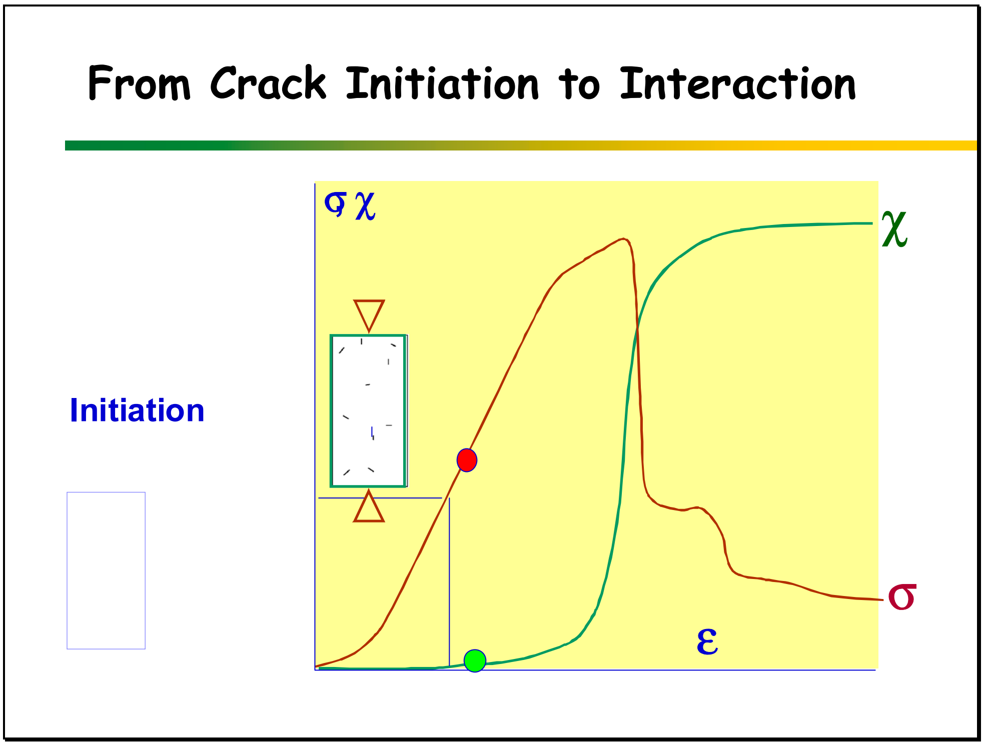


Figure 30

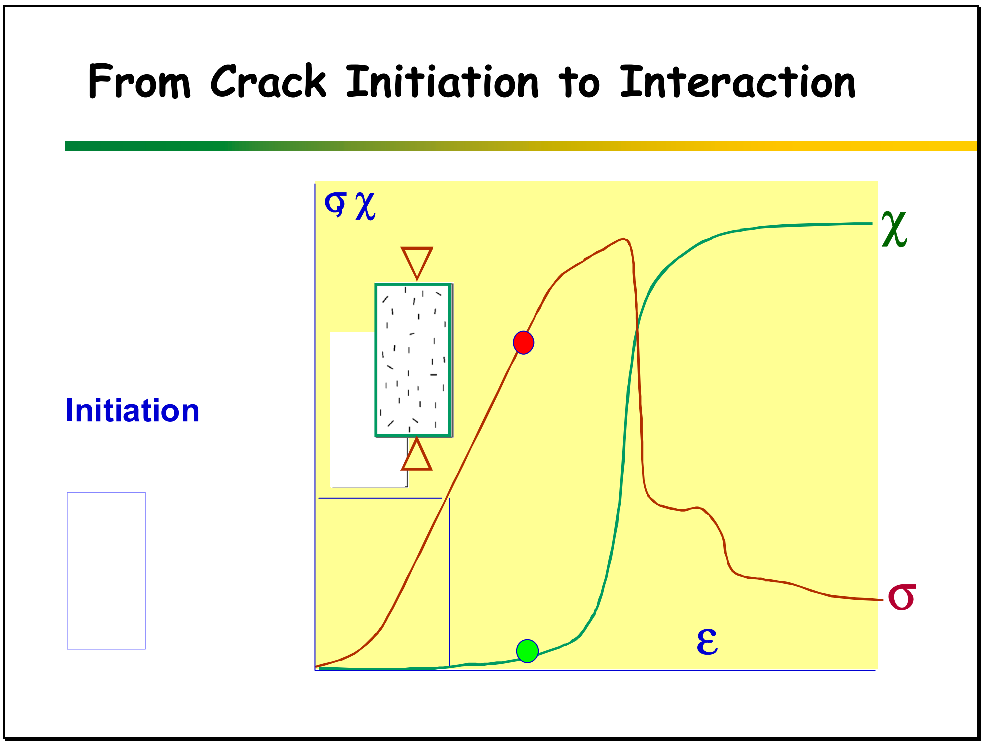


Figure 31

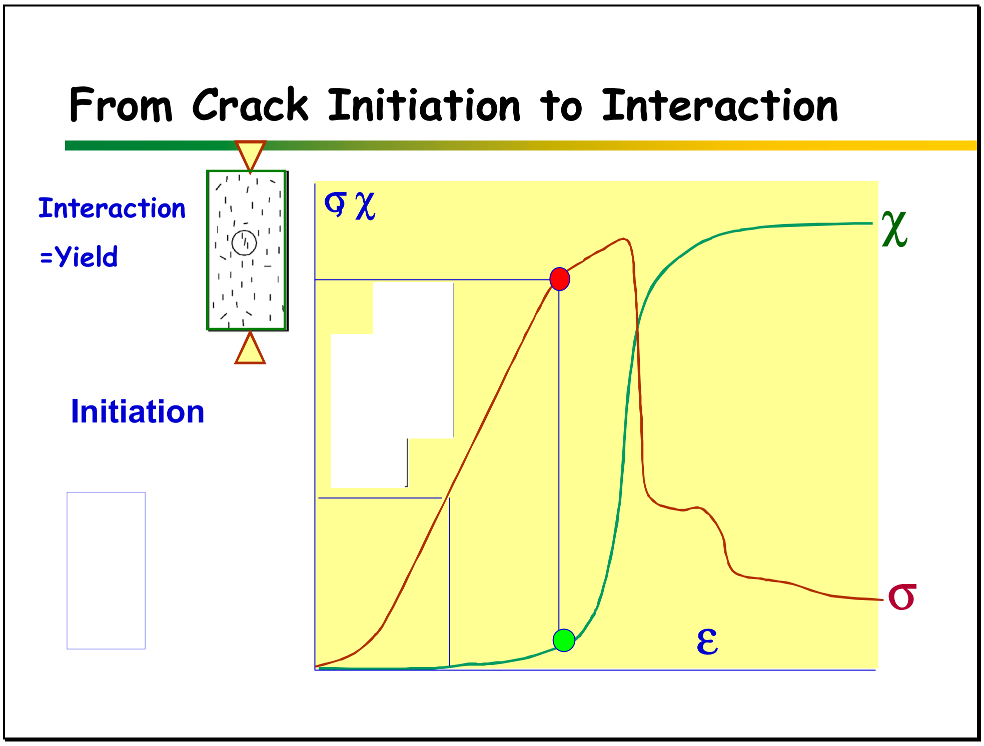


Figure 32

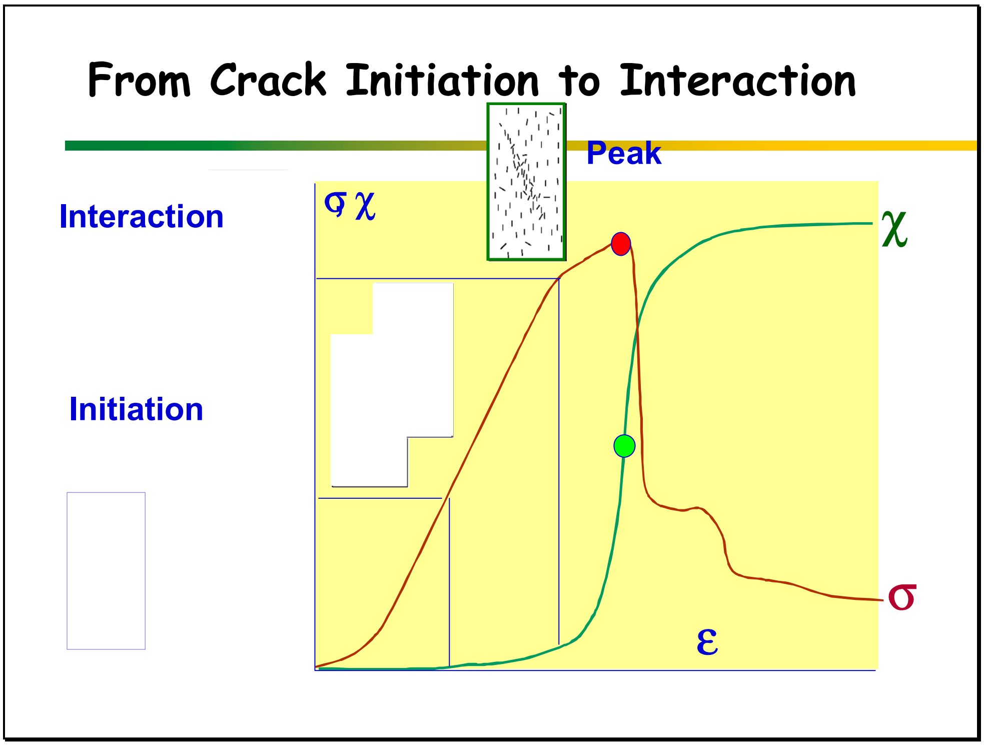


Figure 33

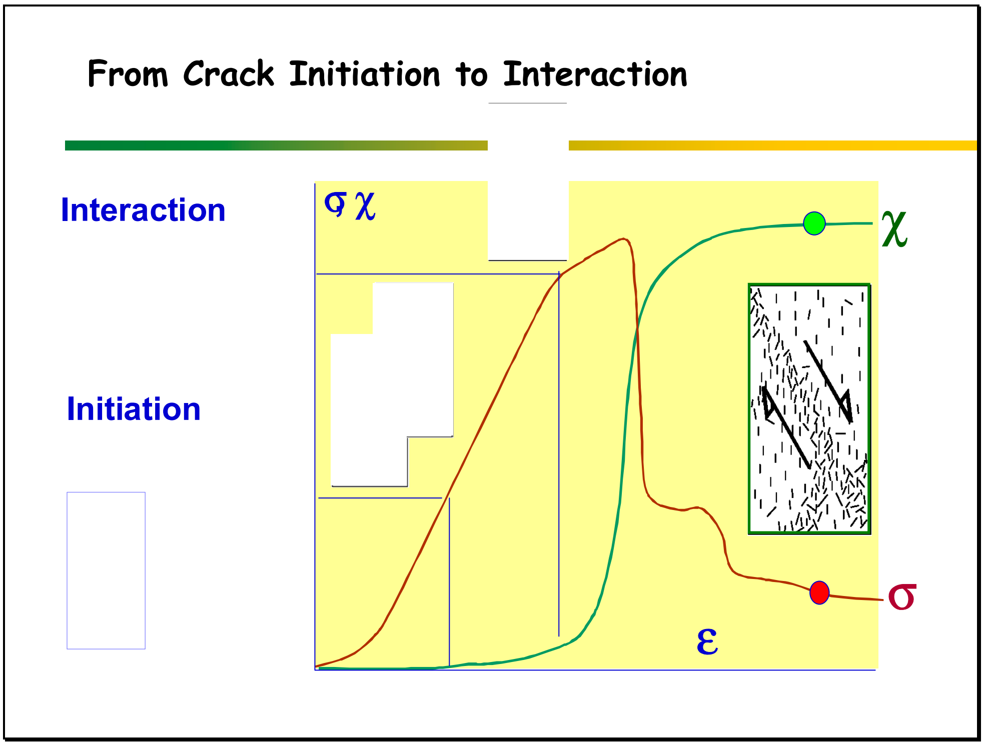


Figure 34

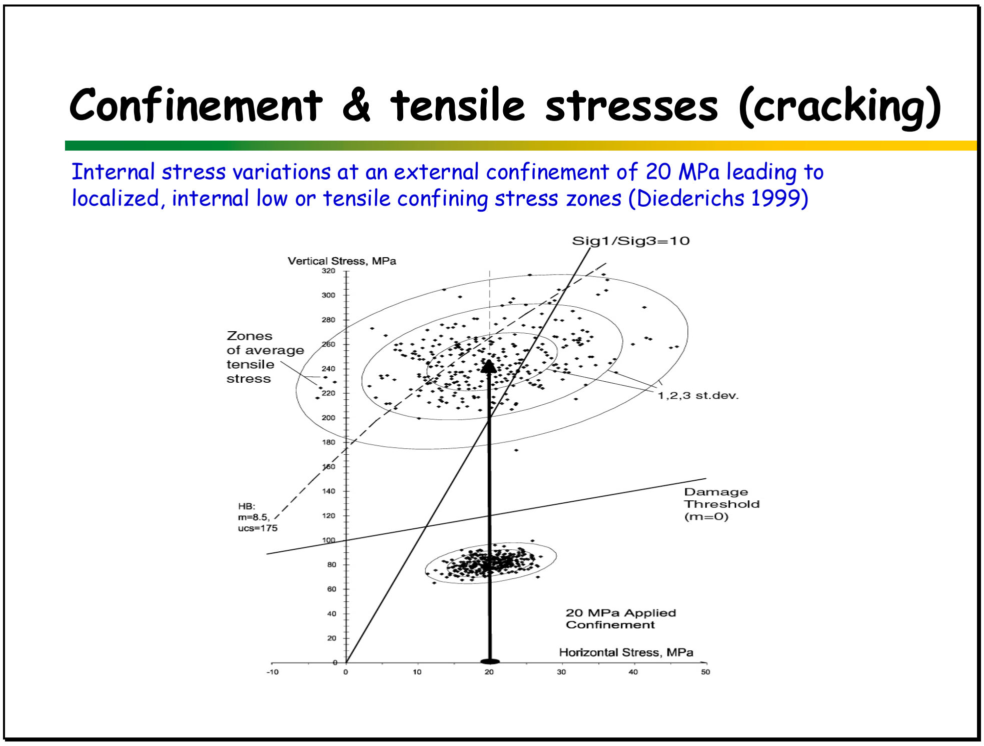


Figure 35

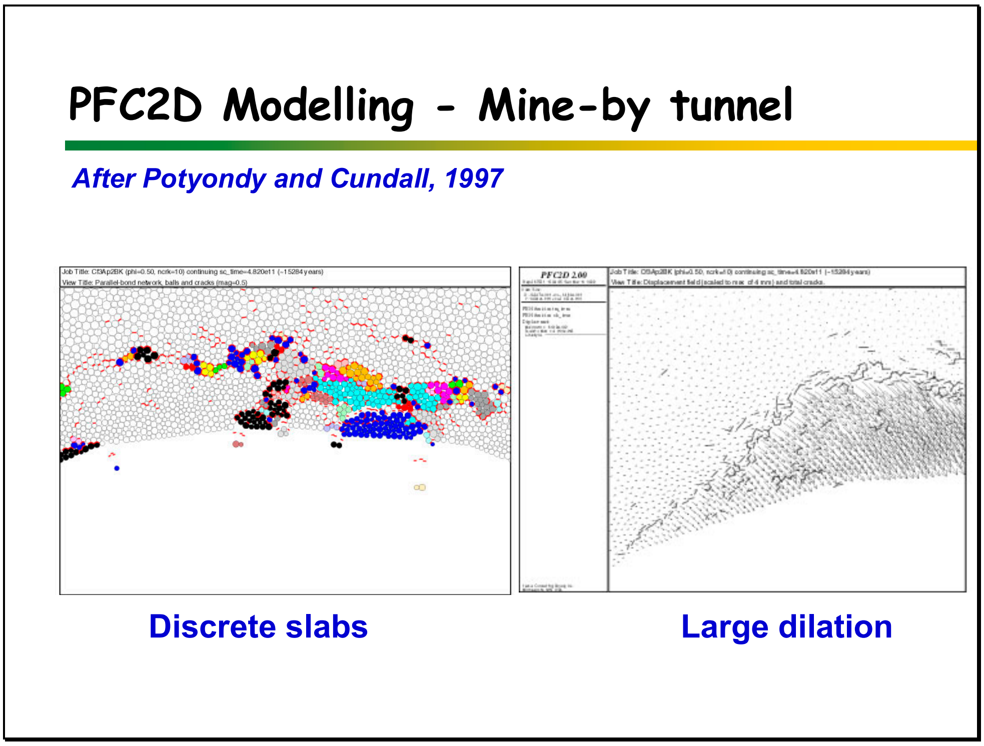


Figure 36

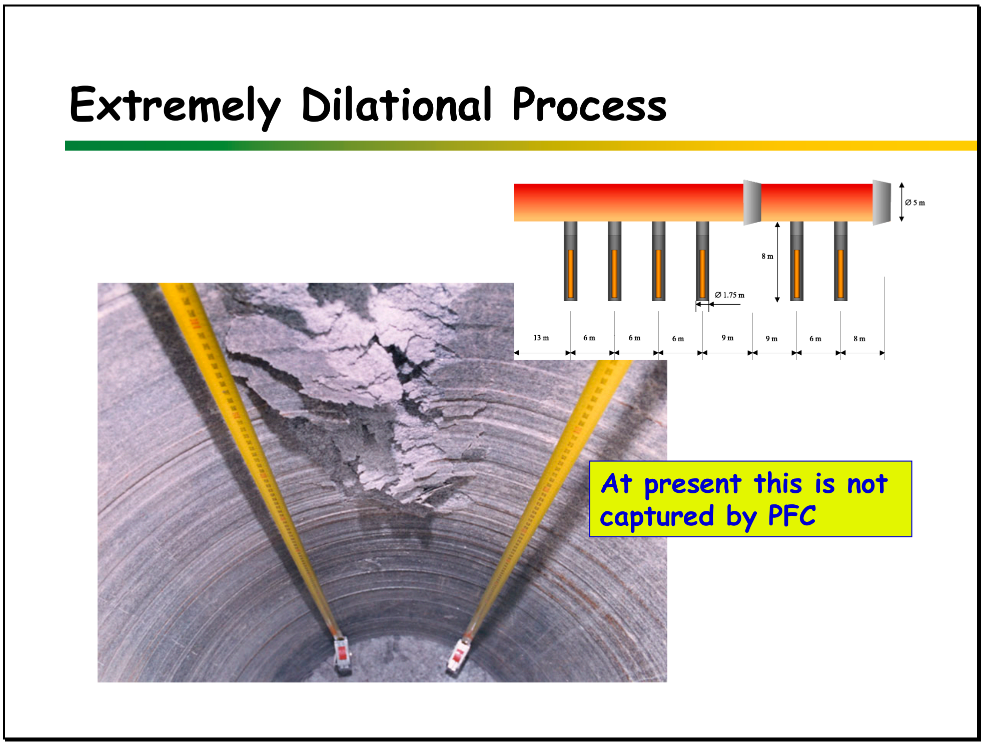


Figure 37

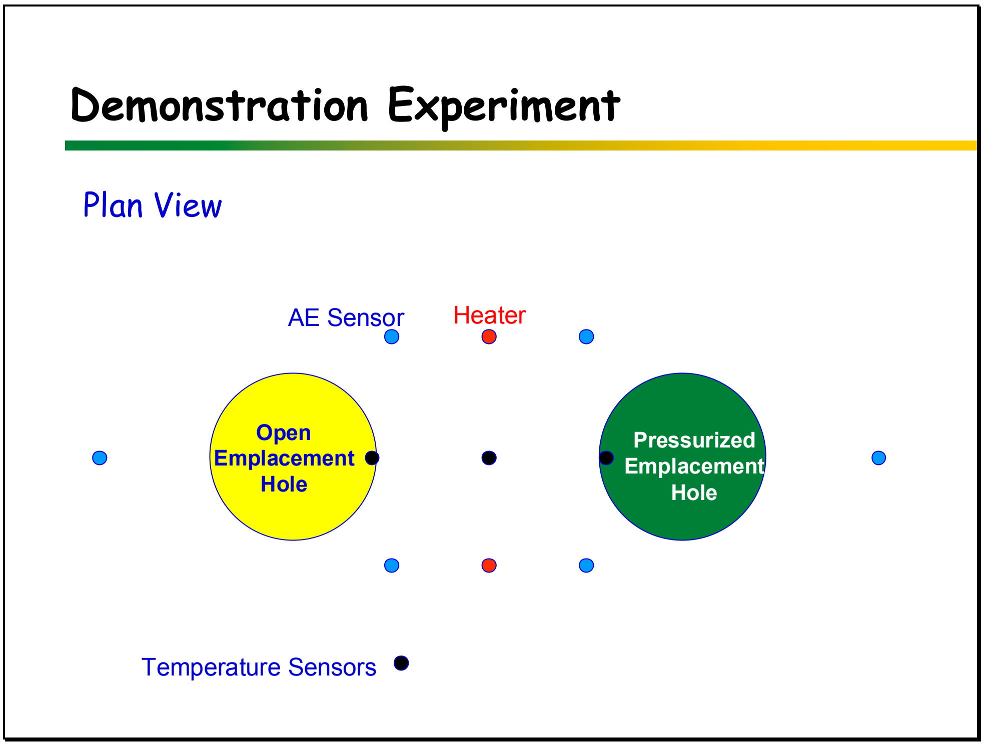


Figure 38

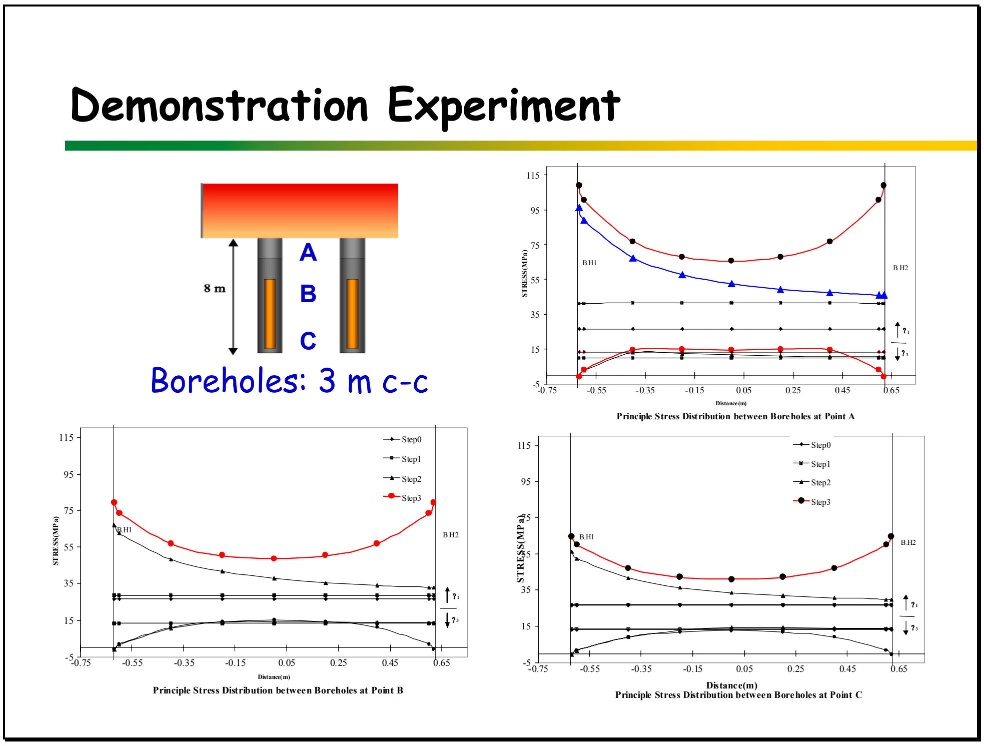


Figure 39

Demonstration Experiment

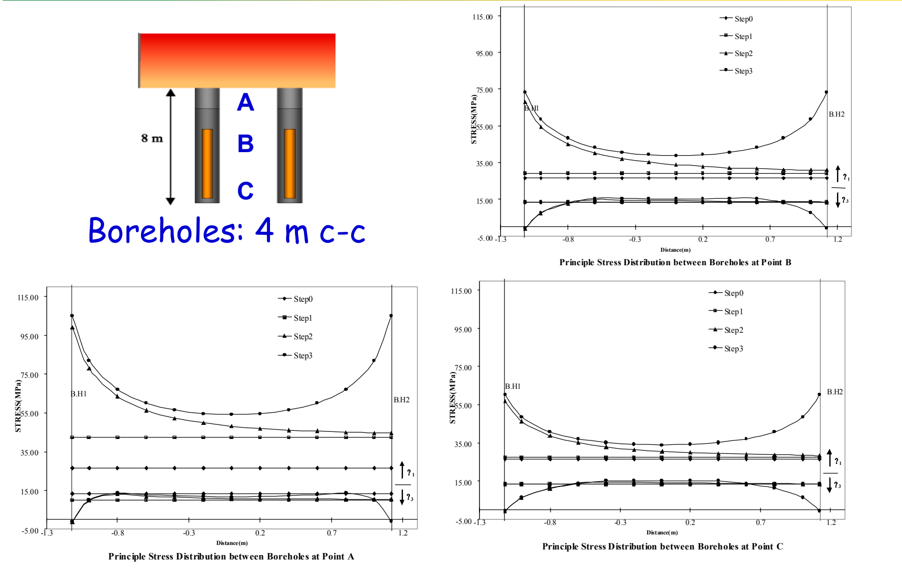


Figure 40

Conclusions

- Practical experience shows that brittle fracture is very sensitive to low (<100 kPa) confining stress.
- Analytical models show that fracture growth is somewhat sensitive to confinement (> 2MPa)
- PFC will be used to explore the effect of confinement on the development of crack interaction and local tensile stresses
- A Demonstration Experiment is proposed to quantify the effect of confinement on brittle fracture.

Paper 10

Acoustic Emission measurements relating to triggering/stabilising microcracking

Paul Young

Cancelled