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Creep in jointed rock masses

State of knowledge

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Preface

This report presents a literature review that focuses on investigations that assess creep in discontinuities. The aim of the study was to find support for a proposed method for bounding estimates of creep movements around openings in a repository, by the use of distinct element codes with standard built-in elasto-plastic models.

A first draft of the report was completed by the end 2003. However, due to other priorities in the repository programme the report was not finalized until 2010. A number of revisions have been made in the current version compared to the original draft and some additional literature has also been included.

The authors acknowledge Philip Curtis, Anders Fredriksson and Derek Martin for examination of the manuscript.

Abstract

To describe creep behaviour in hard rock masses in a physically realistic way, elaborate models including various combinations of dashpots, spring elements and sliders would be needed. According to our knowledge, there are at present no numerical tools available that can handle such a creep model. In addition, there are no records over sufficient long time periods of tunnel convergence in crystalline rock that could be used to determine or calibrate values for the model parameters.

A possible method to perform bounding estimates of creep movements around openings in a repository may be to use distinct element codes with standard built-in elasto-plastic models. By locally reducing the fracture shear strength near the underground openings a relaxation of fracture shear loads is reached. The accumulated displacements may then represent the maximum possible effects of creep that can take place in a jointed rock mass without reference to the actual time it takes to reach the displacements. Estimates based on results from analyses where all shear stresses are allowed to disappear completely will, however, be over-conservative.

To be able to set up and analyse reasonably realistic numerical models with the proposed method, further assumptions regarding the creep movements and the creep region around the opening have to be made. The purpose of this report is to present support for such assumptions as found in the literature.

Summary

This study aims at exploring very slow creep processes that take place not only in the highly stressed rock at the periphery of deposition holes and tunnels, but also a few tunnel diameters farther out in the surrounding rock mass. Slow creep deformations may be of potential importance for direct damage of canisters and permeability effects.

Creep movements around a deposition tunnel in a hard rock mass may be manifested as creep of the intact material and creep along the discontinuities. In a hard rock mass, at normal temperature, it is usually considered that creep deformation in fractures predominates over creep arising in intact rock. The reason is that the shear strength of a fracture is generally less than that of intact rock, particularly along those fractures that are coated or filled with material.

Previous handling of creep in SKB performance assessments was based upon rough bounding estimates and an equivalent continuum representation of the rock mass. Assuming that the rock mass is allowed to creep for an infinitely long period of time, without restrictions, the ultimate result is a hydrostatic stress state in the rock mass adjacent to the deposition tunnel. However, to assume a hydrostatic stress state at a regional scale is unrealistic, considering present global in situ stress observations in crystalline rock /Amadei and Stephansson 1997, Heidbach et al. 2008/. There is also tectonic evidence that supports the view that a threshold stress exists in crystalline rock /Damjanac and Fairhurst 2010/. Hence, in reality creep will not proceed until all rock shear stresses have been completely relaxed, neither adjacent to the tunnel opening nor at regional scale.

To get support for a more realistic numerical modelling technique a literature survey has been performed. The focus of the literature review is on the investigations that assess creep in discontinuities, while creep in intact rock is only dealt with very briefly. The following have been the main key questions:

- are there threshold fracture shear stress/strength ratios, such that no creep takes place unless the threshold ratio is exceeded?
- are there threshold differences between different types of fractures?
- are there potential creep regions around openings, beyond which no creep movements occur, even if the threshold stress/strength ratio is exceeded?

Reported creep tests on clean and filled discontinuities indicate that it is mainly in filled discontinuities, and principally clay filled joints that creep movements have to be accounted for. The laboratory tests reviewed include uniaxial- and triaxial tests as well as direct shear tests. The direct shear tests were performed using constant normal load conditions, while the state around a tunnel is closer to shearing with constant normal stiffness.

The studied results show the occurrence of threshold values, related to the applied shear stress/strength ratio, such that no creep takes place unless the threshold ratio is exceeded. A stress/strength ratio of 30% for unfilled discontinuities and a ratio of 10% for filled discontinuities seem to be reasonable and conservative estimates.

According to the literature it is likely that creep regions with differing extents may arise around a deposition tunnel. An outer perimeter may be determined at a distance of 6 radii from the opening by the stress redistribution.

Recommendations for future work, are to refine previous numerical modelling and to narrow the bounding estimates of creep in the rock mass around a deposition tunnel, based on facts revealed by the literature survey.

Sammanfattning

För att kunna beskriva kryprörelser i hårda berg massor på ett verkligt sätt behövs komplicerade modeller som omfattar flera olika kombinationer av dämpare, fjädrar och förskjutande element. Enligt vår vetskap finns det i dagsläget inte några numeriska verktyg som kan hantera en sådan krypmodell. Dessutom finns det inga observationer av konvergens i tunnlar i kristallint berg över tillräckligt långa tidsperioder som kan användas för att kalibrera värden för modellparametrar.

En möjlig metod för att gränssätta kryprörelser kring öppningar i ett slutförvar är att använda distinkta element program med elasto-plastisk materialmodell. Genom att reducera skjuvhållfastheten för sprickplanen i närheten av förvarets hålrum uppnås en avlastning av skjuvspänningen. Den ackumulerade förskjutningen kommer då att motsvara en övre gräns för kryprörelsen som kan uppkomma i en uppsprucken bergmassa, men utan hänsyn till tiden det tar att uppnå denna förskjutning. Uppskattningar som är baserade på resultat från analyser där skjuvspänningarna tillåts att försvinna helt kommer dock att vara för konservativa.

För att möjliggöra analyser med någorlunda realistiska numeriska modeller med den föreslagna metoden, måste ytterligare antaganden avseende kryprörelsen och krypområdets storlek göras. Rapporten presenterar stöd för sådana antaganden som förekommer i litteraturen.

Contents

1 Introduction

1.1 Importance of time-dependent rock deformation for the deep repository

1.1.1 Construction

During construction there will be different kinds of local instabilities in the walls of tunnels and deposition holes. These instabilities may lead to alterations of the geometry of the openings, e.g. because of intersections of fractures and fracture sets that may promote wedge formation and eventually lead to loosening of individual blocks. If the initial stresses are high, there may also be brittle progressive intact rock failure close to the walls. Most processes that occur during the operational phase can be handled using mining experience and engineering practice. Brittle failure and falling blocks are important for costs and decisions of a practical nature, e.g. on bolting and shotcrete, and for the safety of personnel working underground. These types of failures, in the immediate vicinity of the walls of the excavated rooms, are time-dependent in the sense they do not necessarily have to occur in direct conjunction with the excavation work. However, they are not time-dependent in the sense of this study, which aims at exploring very slow processes that take place not only in the highly stressed rock at the periphery of deposition holes and tunnels, but also farther out in the surrounding rock mass.

1.1.2 Long-term performance

For long-term performance and safety, slow time-dependent deformations may be of potential importance. Time-dependent deformations may occur in the far-field and in the near-field. In this study, the main concern is deformations that occur in the near-field. For safety, there are two main issues to consider: direct damage of canisters and permeability effects.

Direct damage of canisters

There are two mechanisms that need to be considered:

- General convergence of deposition holes. This will compress the bentonite buffer and increase the swelling pressure, which depends strongly on the dry density of the bentonite. If the convergence is very significant then, at least theoretically, the swelling pressure may exceed the canister design pressure.
- Creep-shear deformations along fractures that intersect deposition holes. If the shear displacement exceeds a threshold value the canister counts as failed. At present, the threshold value is 0.1 m.

In reality, creep-induced changes of deposition hole geometries may depend on both mechanisms.

Permeability effects

Convergence of excavation peripheries means that apertures of fractures in the surrounding rock may increase. This will cause the permeability to increase. The extent of that permeability increase depends on the amount of convergence, on how far into the rock the effects extend, and also on the fracture arrangement. For example, small aperture increases on a large number of fractures do not give the same effects as large aperture increases on a small number of fractures. In contrast to mechanical effects, there are no general criteria for acceptable and unacceptable permeability changes.

1.2 Some general aspects on the concept of creep

Creep is defined as the time-dependent deformation of rock under a load that is less than the short-term strength of the rock. Creep strain can seldom be recovered fully when loads are removed, thus it is largely plastic deformation.

1.2.1 Creep stages

Rock creep is usually expressed as a creep function relating strain to time. In its most general form, this creep function can be expressed as /Jaeger and Cook 1976/:

$$\varepsilon = \varepsilon_1(t) + v_2 t + \varepsilon_3(t) \qquad (1\text{-}1)$$

where, ε is the total creep strain; $\varepsilon_1(t)$ is the primary or transient creep, v_2t the secondary or steady-state creep, and $\varepsilon_3(t)$ the tertiary creep or accelerating creep which occurs just prior to failure of the sample. The general form of these three stages is illustrated in Figure 1-1. In the primary stage, the creep velocity decreases steadily with time, in the secondary stage the creep velocity is constant with time, and finally in the tertiary stage the creep velocity increases with time.

Any of the three stages described above may dominate the deformation process, depending on the material and the testing conditions. The schematic curve in Figure 1-1 illustrates a case where the applied load is:

- sufficiently high to take the rock sample through all creep stages,
- constant over time.

Here, "sufficiently high" means that the load constitutes a sufficiently high fraction of the short-term strength. If that fraction is low, then it may not even suffice to take the rock sample into the secondary creep stage, in which case the creep movement will slow and eventually die out.

Creep movements around openings in the repository do not take place under constant load conditions. The effect of creep movements is to reduce the stress anisotropy such that rock volumes, or parts of individual fractures that have entered stage II, are likely to fall back into stage I.

1.2.2 Creep laws

Most formulations of creep in rock suggested in the literature can be separated into two main categories:

1. empirical creep functions, based upon curve fitting of experimental data.
2. rheological creep functions, based upon creep behaviour models composed of assemblages of elastic springs, viscous dashpots, plastic slider and brittle yield elements. .

Common forms of these two types of creep functions are described by /Dusseault and Fordham 1993/ and /Ladanyi 1993/.

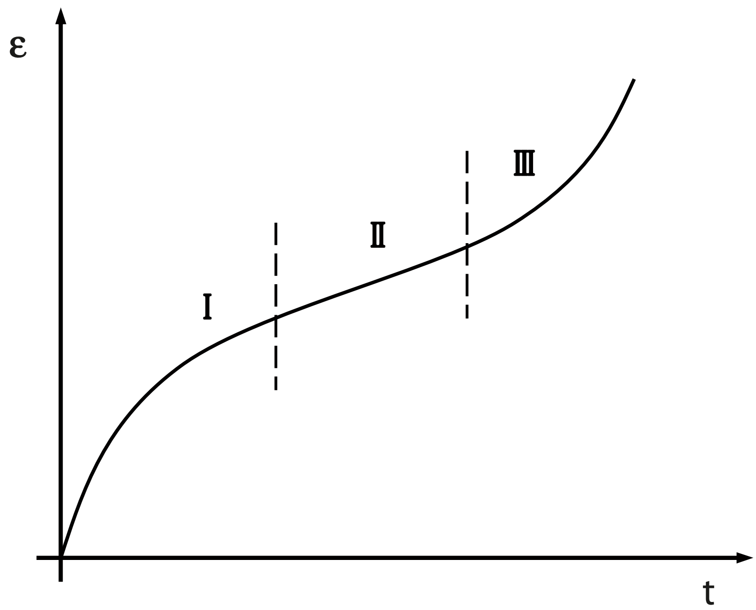

Figure 1-1. *Idealized creep curve showing axial strain versus time including the three major creep stages: I) primary creep, II) secondary creep, and III) tertiary creep.*

Empirical creep functions

The most common empirical creep functions are the power law:

$$\varepsilon = Bt^{n} \qquad 0 < n < 1 \tag{1-2}$$

and the logarithmic creep function:

$$\varepsilon = A \ln t \tag{1-3}$$

in which *A, B* and *n* are constants. All empirical creep functions can be shown to be related if they are expressed in terms of creep strain rate:

$$\dot{\varepsilon} = Ct^{-M} \qquad 0 \leq M \leq 1 \tag{1-4}$$

Steady state secondary creep corresponds to $M = 0$ and $\dot{\varepsilon} = C =$ constant. If $0 < M < 1$, the power law creep function (Equation 1-2) is obtained. $M = 1$ gives the logarithmic law (Equation 1-3). Values for M less than zero imply tertiary creep.

Rheological models

The simplest rheological creep model for primary or transient creep is the Kelvin model, which consists of spring element and a viscous element (dashpot) in parallel:

$$\varepsilon = \frac{\sigma}{K}\left[1 - e^{-tK/\eta}\right] \tag{1-5}$$

where σ is the stress and t is time. K is the spring constant and η is the dashpot viscosity. Note that the Kelvin model does not include any plasticity: some time after the load has been removed the strain will be fully recovered.

To model primary and secondary creep stages, the Burghers rheological model is often employed. The Burghers model is a Kelvin element in series with a second dashpot and a second spring element:

$$\varepsilon = \frac{\sigma}{K_1}\left[1 - e^{-tK/\eta_1}\right] + \frac{\sigma}{\eta_2}t + \frac{\sigma}{K_2} \tag{1-6}$$

To describe real creep behaviour in hard rock masses, more elaborate models including various combinations of dashpots, spring elements and sliders would be needed. According to our knowledge, there are at present no numerical tools available that can handle such a creep model. In addition, there is no experimental data e.g. recordings over very long time periods of tunnel convergence in crystalline rock that could be used to determine or calibrate parameter values for such a model.

1.3 Previous handling of creep in SKB performance assessments

In the SKB safety analysis SR 97 /SKB 1999/, effects of time-dependent deformations were considered mainly from the canister damage point of view. The main issue was the concern that convergence of deposition holes may compress the bentonite buffer and thereby increase its swelling pressure, eventually producing loads on the canister that exceed the design pressure. It was concluded that this process, i.e. creep-induced convergence of deposition holes, cannot lead to canister failures.

The conclusion in SR 97 was supported mainly by bounding estimates. If the rock mass is allowed to creep for an infinitely long period of time without restrictions, the ultimate result is a hydrostatic stress state. The deposition hole convergence and the accompanying compression of the buffer, would continue until the buffer swelling pressure has increased sufficiently to balance the hydrostatic rock pressure. The pressure transferred to the buffer would be the mean compressive stress at repository depth, i.e. about 20 MPa. If the rock mass behaves as a viscous medium on the repository scale, not only locally, then the compressive mean stress would be smaller, about 14 MPa, since this is the hydrostatic rock pressure that corresponds to the rock overburden at 500 m depth.

The bounding estimates are very conservative. In reality, creep will not proceed until all rock shear stresses have been completely relaxed. Consequently, the stresses that may be transferred to the buffer and the canisters as a result of rock creep will be smaller. For intact rock, experience says that there are threshold values of rock shear stresses that mean that creep movements die out when the shear stresses approach or drop below that level /Pusch and Hökmark 1992/.

1.4 Key questions

In future safety assessments more realistic, but still conservative, ways of estimating time-dependent convergence of deposition holes and tunnels may be needed.

The previous safety assessment method was based upon an equivalent continuum representation of the rock mass. In reality, the mechanical behaviour of typical rock masses is determined by the presence of discontinuities. That this also holds true for creep deformations is a logical assumption or generalization that will be verified in the following chapters. It is shown, for instance, that creep strain in intact crystalline rock gives modest net deformations even after very long time periods, provided the shear stress does not constitute a substantial fraction of the intact rock shear strength.

The above means that it is reasonable to neglect the intact rock component of creep strain and assume that all time-dependent rock mass behaviour is a result of time-dependent fracture behaviour.

A way to find bounds to rock mass creep effects in the near-field, may be to analyse discrete fracture numerical models of tunnels and deposition holes and to assume that all shear stresses on all fractures disappear over time. The fracture movements associated with that stress relaxation would give an upper bound estimate of the ultimate possible rock mass deformation. Such a modelling attempt has been done by /Glamheden et al. 2001/ using the 3DEC code. Estimates based on results from analyses where all shear stresses are allowed to disappear completely will, however, be over-conservative. For instance, it is likely that there are threshold stress/strength values, such that creep movements die out when fracture stress/strength ratios drop below that value. It is also likely that there is no tendency for creep in rock regions that are not affected by the presence of openings or by thermal or hydrological disturbances. To be able to set up and analyse reasonably realistic numerical models, assumptions regarding these issues have to be made. Support for such assumptions is sought for in the literature in this study. The following are the main key questions:

- Are there threshold fracture shear stress/strength ratios, similar to threshold ratios found for intact rock, such that no creep takes place unless the threshold ratio is exceeded?
- Are there threshold differences between different types of fractures?
- Are there potential creep regions around openings, i.e. can some outer perimeter be found, beyond which no creep movements occur, even if the threshold stress/strength ratio is exceeded?

1.5 Scope of work

A large number of papers and reports have been published concerning creep in rock masses. Authors working with different goals and sometimes also using different vocabularies have written them. These circumstances lead to the term ‘rock mass creep’ having different meanings in different papers. As a matter of fact, the main part of the literature dealing with creep in hard rock masses is focused on the creep behaviour of intact rock.

In this survey, the focus is on the limited number of investigations that actually concern creep in discontinuities. The main issue is the problem of threshold creep stress-strength ratios for discontinuities. Creep in intact rock is dealt with very briefly in Chapter 3.

The literature review is also aimed to cover information that may be relevant to the problem of defining creep regions around excavations, i.e. regions in which creep along fractures is likely to occur if the stress-strength ratio exceeds the threshold.

2 Creep in rock masses

2.1 Introduction

Time-dependent deformations in a hard rock mass may be manifested as creep of the intact material and /or creep along the discontinuities. In intact crystalline rock at normal temperatures creep is primarily the product of time-dependent micro fracturing of the rock; which will produce both shear and volumetric strains if the rock is dilatant /Schwartz and Kolluru 1981, Rummel 1982/. The principal factors controlling creep rates in intact rock are applied deviator stress, effective confining pressure and moisture content /Kirby and McCormick 1984/. Creep in rock joints occurs as normal compression and shear movements along the discontinuities. Factors that govern the creep rates in discontinuities are the character of the joints and the geometry of the joint system with respect to the excavation /Ladanyi 1993/. The total time-dependent response of a rock mass depends on the relative contribution from each source.

It is usually considered that creep deformations in fractures overshadow intact rock creep in hard rock masses. The reason is that the shear strength of a fracture generally is less than that of intact rock, in particular along those that are coated or filled with material /Bowden and Curran 1984, Höwing and Kutter 1985/. The movements will arise along the joint system with most unfavourable conditions and direction with respect to the excavation. In rock masses with high deviatoric in situ stresses and a sparse, interlocked fracture system, a possibility exists that intact rock creep could dominate.

In the present chapter a brief generic overview will be given of the research into the topic of creep movements of discontinuities in hard rock masses. The work of a selected number of researchers who have made valuable contributions to the understanding of the creep in fractures is described.

The investigations reported in the literature have been classified into two groups, depending on whether the work mainly deals with time-dependent strength or time-dependent displacement of discontinuities. In addition, the latter group is further subdivided to distinguish laboratory tests of unfilled joints from tests of filled joints, which according to published results exhibit large differences in creep behaviour.

The applicability of the test results for estimating the scope of creep in the repository host rock is briefly discussed in this chapter. A more comprehensive description of results considered to be of interest is found in Chapter 4.

2.2 Time-dependent shear strength

Early research work on rock creep was concentrated on the time-dependent post-failure strength of originally intact rock /Bieniawski 1970/ and jointed samples /Kaiser and Morgenstern 1979/. This topic is not considered to be relevant under the stress and strain conditions encountered in a repository. This work is therefore not described further in this report.

A number of studies of time-dependent shear strength in discontinuities have been related to the investigation of earthquake mechanisms, focusing on time-dependent shear strength in deep faults /Dieterich 1972/. Other applications for studies of the time-dependent shear strength in discontinuities have been the stability of rock slopes and long-term strength of hard rock masses /Lajtai and Gadi, 1989, Lajtai 1991/. The investigations show that under certain conditions the frictional resistance of a discontinuity may increase and impede the development of steady-state creep. This fact is of some importance for the current review and a description of the studies is given below.

/Dieterich 1972/ performed direct shear tests on greywacke, granite, quartzite and porous sandstone. His investigations focused on the relationships between the duration of stationary contact and the static friction coefficient of unfilled and gouge filled rock joints. Samples were prepared by sawing blocks with subsequent lapping to obtain flat and parallel slip surfaces of a required roughness. The duration of stick was varied at a constant normal stress by holding the shear stress slightly below the peak strength. The static friction at the end of a desired interval was then measured by rapidly increasing the shear stress until the block moved. The time interval was between 1 sec and 24 hours and the normal stress was varied between 2 and 85 MPa. The results show that the coefficient of static friction of joints is time independent for a clean rough joint surface while a joint with gouge exhibits a highly time-dependent behaviour. Static friction increases with the time that adjacent blocks remain in stationary contact. Dieterich suggests that compaction of the gouge separating the slip surfaces is time controlled and determines frictional strength. One implication of the work is that, in gouge filled joints mainly primary creep should be expected, since the joint friction coefficient increases with time.

/Lajtai and Gadi 1989/ studied the time-dependence on friction in fractures planes. They performed direct shear tests on smooth, planar rock blocks of Lac du Bonnet granite. The test specimens were cut by diamond saw and ground to the required size with subsequent polishing of the shear surface. The sample size amounted to 40 mm by 90 mm. The maximum displacement amounted to 500 mm achieved by repeated shearing of the same surface in steps of 25 mm. The normal load ranged from 0.2 to 8 MPa. No steady state displacement was observed in any of the tests. The frictional resistance in the tests increased with both displacement and time. The increase of the frictional resistance of an initially smooth and polished surface during continuing shearing displacement is due to wear. The results confirm the findings by /Dieterich 1972/ that most likely creep is transient in rock joints under shear conditions that accumulate gouge fill.

2.3 Time-dependent displacements

2.3.1 Creep in unfilled joints

Creep movements in unfilled rock joints have been studied by a number of workers with reference to rock engineering and design of underground structures. Uniaxial tests and direct shear tests have been performed to investigate creep in fractures close to the ground surface /Amadei and Curran 1982, Bowen and Curran 1984, Schwartz and Kolluru 1981, 1982/. Triaxial tests have been performed to investigate creep in deep seated fractures and faults under high confining pressure /Wawersik 1974, Solberg et al. 1978/. The work of the above mentioned researchers is described below.

/Amadei and Curran 1982/ conducted triaxial and direct shear tests on unfilled and clean rock joints under a variety of stress states and surface conditions. The rock types tested were oven dried (120°C) sandstone, limestone, marble and granite. Triaxial creep tests were performed on artificially jointed cylindrical samples of approximately 54 mm diameter (NX size), with a diamond saw cut at a 30-degree angle with respect to the axis of the sample. A few intact rock samples were also tested in order to separate creep of intact rock from creep along the joint. The direct shear tests comprised samples 127 mm by 127 mm in size, including an artificially prepared horizontal saw cut. Stresses used in the experiments ranged up to 9.5 MPa normal stress and 5.7 MPa shear stress in the triaxial tests and 0.5 MPa normal stress and 0.3 MPa shear stress in the direct shear tests. Creep displacements were recorded over periods ranging from 5 to 300 hours. In general, the creep behaviour of unfilled discontinuities was found to be similar to that of an intact rock. Of special importance for the current study is that the results show the occurrence of a threshold value, related to the applied shear stress – shear strength ratio. Beyond a ratio of approximately 0.5 the creep movement increases greatly for a joint in a marble sample.

/Bowden and Curran 1984/ investigated the creep behaviour by direct shear tests on artificially prepared fractures in clastic shale. The mineralogical composition of the shale consisted primarily of clay minerals (illite and chlorite, 75–90%) with minor amounts of crystalline silicates (5–20%) and lesser amounts of calcite, dolomite, and heavy minerals. The shale blocks were cut parallel to the bedding and lapped until planar and uniform. The sample size was 200 mm by 200 mm. Factors studied were the relative responses of sheared and unsheared joints to normal loading and stress relief, and the influence of time-dependent displacements of constant normal and sub-critical shear stresses. The test duration was typically 4–5 days, with a normal stress of 1 or 2 MPa and shear stresses maintained constant throughout the tests. The most important result for the current study is the observation of a threshold value, related to the applied shear stress/shear strength ratio. The determined curves display a creep rate, which is relatively insensitive to changes in stress/strength ratios below 0.7, and which decreases rapidly with time. The value of the ratio is higher than the value presented by /Amadei and Curran 1982/. The divergence probably depends on the distinction in discontinuity roughness between a sample in marble and a sample in shale. The conditions of a discontinuity in marble are most likely closer to that of hard rock.

/Schwartz and Kolluru 1981, 1982/ investigated the influence of shear stress levels on the creep of artificial cohesive and non-cohesive joints. Uniaxial creep tests were conducted on intact samples of gypsum plaster and samples with discontinuities inclined at angles between 0 and 60 degrees to the axial stress. The samples were rectangular, 32 mm by 32 mm by 121 mm in dimension, with a single joint through its midpoint. Samples with a joint with small cohesive strength were produced by casting specimens in a mould with a PVC insert with an inclined face. The liquid plaster was poured into one half, vibrated, and allowed to set. The insert was then removed and replaced with more liquid plaster cast directly against the first half. Samples with a non-cohesive joint were produced by casting the two halves of the specimens as individual blocks and then stacking one on top of the other. The tests with jointed specimens were conducted at a stress level up to 0.4 σ_c over a period of approximately 60 hours. Although a synthetic rock material was used, the findings from the tests are of interest, since they bring about a general understanding of the time-dependent mechanism in fractures. /Schwartz and Kolluru 1982/ present a simple theoretical model that at least in part can explain the observed joint creep behaviour. As the first team of researchers they concluded that the creep appears to depend not only on the applied shear stress to shear strength ratio but also on the absolute shear and normal stress levels across the discontinuity. /Malan et al. 1998/ have later confirmed this statement.

/Wawersik 1974/ studied time-dependent deformation on air-dry and water-saturated samples of Westerly granite and Nugget sandstone. Triaxial tests were performed on cylindrical specimens, 25.5 mm in diameter and approximately 60 mm in length. Most tests were on intact specimens, but a few were performed on samples including one artificially induced joint (roughness ±0.5 mm) oriented at a 30° angle with respect to the sample axes. The joints were formed by inducing tensile fractures in a block of rock, gluing this fracture together, coring a cylindrical test sample at a proper orientation, and then dissolving the epoxy glue with solvent prior to testing. The tests were carried out under 20.7 MPa confining pressure with a deviator stress of 60.5 MPa and 77.3 MPa, respectively. Axial and radial strains were measured over periods up to 30 hours. The results show that the creep behaviour of jointed rock has the same character as that of the intact rock, but that the magnitude of strains is much larger. They also demonstrate that the instantaneous joint deformation exceeds the time-dependent response by several orders of magnitude. However, the test period is small when compared to the periods of interest for a repository.

/Solberg et al. 1978/ simulated creep along a fault by testing a cylindrical sample of Westerly granite, 63.5 mm in length and 25.4 mm in diameter, containing a saw cut joint inclined at 30°, filled with gouge of 1 mm thickness from crushed Westerly granite. The stresses applied on the samples were very high, 400 MPa confining pressure and up to 1,080 MPa deviator stress, in an attempt to simulate conditions along a real fault. The results showed that primary creep was followed by constant rate secondary creep at a stress ratio of 0.85 of the joint shear strength. Furthermore, the results show that at a confining pressure of several 100 MPa the crushed gouge filling material becomes densely compact and its shear strength approached that of intact rock at the same confining pressure. The applicability of the findings are limited with regard to the current study, since the stress level in the laboratory test far exceeds the expected stress condition around a repository. In addition, our concern is mainly with threshold values related to primary creep rather than with secondary creep as in the experiment.

2.3.2 Creep in filled joints

Studies of creep movements in filled rock joints have been reported by two teams of researchers. The main purpose of the investigations has been progressive movements of large natural rock slides /Höwing and Kutter 1985/ and creep deformations around deep level mines in hard rock /Malan et al. 1998/. The work of these studies is described below.

/Höwing and Kutter 1985/ carried out direct shear tests with smooth saw-cut rock surfaces, and a few samples of tensile fractures. The rock type used in all cases was medium grained sandstone. The aim of the study was to determine those factors that predominantly influence the creep behaviour of filled fractures. The factors varied in the tests were: type of filling material, width of the filling layer, normal and shear stress, and sample size. The filler material selected for the creep tests included clay, silt and sand. The filler width ranged from 1 to 10 mm and the sample size was 100 mm by 100 mm. The period of a test lay between 2 hours and 7 days with a normal stress generally between 1 and 2 MPa. The results, which are considered to be of great interest for the current study, show that the creep rate of a filled rock discontinuity is primarily determined by the particle size distribution. The percentage of clay in the filler is much more critical for the creep velocity than the filler width.

/Malan et al. 1998/ performed laboratory investigations of the creep in fractures in hard rock. Unfilled mining-induced tensile fractures in lava, along with gouge-filled saw cut discontinuities in quartzite, were examined in direct shear tests. The gouge material used in the tests was natural as well as artificially obtained by crushing quartzite. The particle size distribution of the gouge filling used corresponds to medium to fine grained sand. Factors that were studied were the effect of the shear stress to shear strength ratio (τ/τ_s), the effect of the magnitudes of the shear and normal stresses, and the effect of gouge thickness. The tests were conducted using a normal stress of 0.5, 1.0 and 1.5 MPa. The shear stress was increased in a stepwise fashion with a period of 24 or 48 hours between load increases. The test duration was up to 5 days. Although Malan et al. have concentrated the investigation on steady-state creep and our concern is mainly with primary creep the findings are considered valuable. The results clearly show that the primary and secondary creep rate increases with increasing shear stress to shear strength ratio. This creep rate is also affected by the gouge thickness and the absolute values of shear and normal stress. The results further display, that in comparison with gouge-filled discontinuities, the creep behaviour of unfilled fractures in hard rock is negligible.

2.3.3 Long-term displacements measurements in the field

Long-term displacements measurements of hard rock are rare, and those that exist are normally limited to a period of some years, which is a short period relative to the repository life. However, a case of interest is a long-term measurement performed in an access shaft for the Underground Research Laboratory in Canada /Martino 1995/. The shaft, which has a diameter of 4.6 m, was constructed in hard rock of sparsely fractured granite under high in situ stresses. During the construction of the shaft mechanical extensometers were installed to monitor radial displacements. Four of these extensometers, two at approximately 325 m depth and another two at 385 m depth, were left in place to monitor any long-term displacements. The monitoring, which continued for about 5 years, showed declining displacements in most positions and a strain rate less than $3{\cdot}10^{-12}s^{-1}$ in all positions. Although, the monitoring time is short compared to the life of a repository, the results indicate that for this type of rock, with few fractures, the expected rate of creep is very low.

2.4 Summary

The reported results display large differences in the creep behaviour of clean and filled discontinuities. The creep movement in a rough fracture seems to be negligible in comparison with a filled discontinuity /Malan et al. 1998/.

A number of investigations indicate that the creep behaviour of an unfilled discontinuity has the same character as that of the intact rock, however, the magnitude of strain is somewhat larger /Amadei and Curran 1982, Wawersik 1974/. The instantaneous deformation in tests of unfilled joints exceeds the time-dependent response by several orders of magnitude.

Investigations of filled discontinuities show a creep rate that is dependent on the filling thickness and the particle size distribution of the filling material. The percentage of clay in the filler seems to be much more critical to the creep velocity than the filler width /Höwing and Kutter 1985, Malan et al. 1998/.

Reported results show the occurrence of a threshold value, related to the applied shear stress to shear strength ratio, beyond which the creep movement in a joint increases greatly for both clean and filled discontinuities /Amadei and Curran 1982, Bowden and Curran 1984, Höwing and Kutter 1985, Malan et al. 1998/, see Table 2-1. According to the test observations the threshold value lies between 0.5 and 0.7. The spread in the values is probably due to differences in the test conditions, i.e. initial conditions, the stress and displacement magnitude, the duration of the tests and the distinction in fracture resistance between different types of rocks and fillings.

The creep rate appears to depend not only on the applied shear stress to shear strength ratio but also on the absolute shear and normal stress level across the discontinuity /Schwartz and Kolluru 1981, 1982, Malan et al. 1998/.

Long-term displacements measurements, in hard rock consisting of sparsely fractured granite under high in situ stresses, indicate creep rates that are very low /Martino 1995/.

Table 2-1. Compilation of test conditions and the evaluated creep threshold value for different investigations.

Reference	Testing method	Sample size (mm)	Rock Type	Joint Type	σ_n (MPa)	τ (MPa)	τ/τ_p
Amadei and Curran, 1982	Triaxial tests	NX	Granite, marble	Artificially	4.7–9.5	1.5–5.7	0.6
Amadei and Curran, 1982	Direct shear tests	127×127	Granite, marble	Artificially	0.3–0.5	0.2–0.3	0.5
Bowen and Curran, 1984	Direct shear	200×300	Shale	Artificially	1–2	–	0.7
Höwing and Kutter, 1985	Direct shear tests	100×100	Sandstone	Filled	1–2	–	0.7[1]
Malan et al. 1998	Direct shear test	100×125	Lava	Filled[2]	0.5–1.5	0.3–0.5	0.5

1) Based on the residual strength.
2) A single unfilled fracture not included here was also tested.

3 Creep in intact rock

3.1 General

The effect of time on the mechanical behaviour of intact rocks has two main origins:

- dislocation of defects in the crystalline structure,
- stable micro fracturing.

Which one of the two processes being the most important for the observed macroscopic rock response depends mainly on factors such as the stress level relative to the rock strength, the temperature relative to the melting temperature and confining pressure relative to the brittle-ductile transition pressure /Landanyi, 1993/.

The subdivision of rock behaviour into brittle, semi-brittle and ductile regimes, based on the stress-strain curves from compressive tests on rock samples, is also generally applied to time-dependent deformation. Under long-term loading, crystalline rocks are considered to be in a brittle state at temperatures below 100–150°C /Pusch and Hökmark 1992, Eloranta et al. 1992/. In this regime, the time effects are considered to be controlled by the rate of the slow growth of microcracks. That creep strains are due to microcracking, is clear from direct measurements of cracks in rocks that have undergone creep and from a large number of indirect measurements, such as acoustic emissions, the velocity of elastic waves and permeability /Kirby and McCormick 1984/.

Crystalline rock with low porosity cannot display significant steady-state creep under brittle conditions /Carter et al. 1981/. The dominant mechanism of micro-crack generation and propagation along grain boundaries, either stabilises or accelerates /Dusseault and Fordham 1993/. Cracks extend from the stress concentrators- cracks, pores, and grain boundaries; and the rate of growth decreases as the number of concentrators decreases by cracking. Thus creep rates decrease with time in a hardening stage. However, if the crack density exceeds some critical value such that cracks are close enough to interact on a large scale, crack coalescence occurs to form a macroscopic fracture on which failure occurs /Kirby and McCormick 1984/.

The often reported decrease in creep rate is illustrated schematically in Figure 3-1, which shows log time creep behaviour for granite, extrapolated from laboratory test results, obtained from uniaxial load tests with axial loads between 1/3 and 2/3 of the uniaxial compression strength /Pusch and Hökmark 1992/. The two curves correspond to two different experiments performed on different rock types and with different loads, giving different values of the parameter A in the log-time creep law. The figure should be interpreted with caution, since extrapolating laboratory test data to long time periods is an uncertain procedure. Here it is sufficient to conclude that the creep strain after 100,000 years seems to be too small to have any potential of influencing the convergence of repository openings.

A brief summary of literature findings on the subject of intact rock creep is given in this chapter. The main factors controlling brittle creep rates, i.e. applied deviator stresses, effective confining pressure and moisture content, are discussed below.

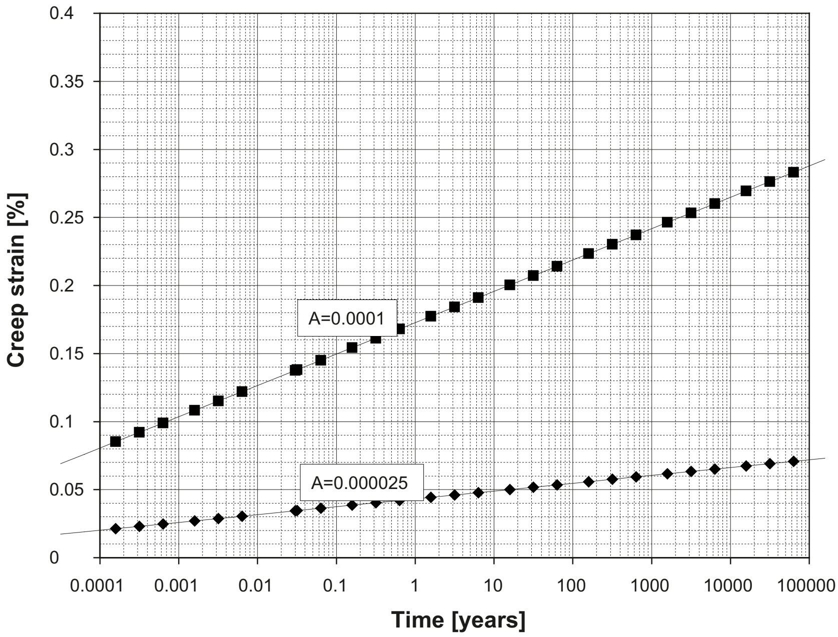

Figure 3-1. *Intact rock creep movements extrapolated from results of uniaxial creep laboratory tests of short duration, assuming creep obeys the creep law $\varepsilon = A*\ln(t)$. For the upper curve (A=0.0001) the axial stress corresponded to 40% of the compressive strength /Pusch and Hökmark 1992/.*

3.2 Influence of deviator stress

Increasing the applied deviatoric stress increases the creep rate. Different rocks require different levels of deviatoric stress to creep at the same rate. It has also been observed that several rocks will not show significant creep unless the deviatoric stress is above some threshold value /Dusseault and Fordham 1993/. According to /Damjanac and Fairhurst 2010/ the threshold deviatoric stress for brittle crystalline rock is about 40–60% of its compressive strength.

3.3 Influence of confining pressure

The effect of an increased confining stress is to decrease the creep rate, because confinement acts to suppress the tension stresses associated with crack growth /Damjanac and Fairhurst 2010/. It is also assumed that the confining pressure increases the energy barrier to be overcome for continuous crack propagation. In addition, high confining pressures result in the closure of cracks, which reduces the transport of fluids and gases through the rock. This has a limiting effect on stress corrosion, and hence on the creep rate, because corrosive agents cannot easily reach new crack surfaces that are successively created during creep /Krantz 1980/.

3.4 Effects of water content

The presence of pore water in rocks is found to affect their behaviour in two ways:

- It enhances the rate of crack formation under stress.
- It produces a temporarily internal confinement.

The effect of the rate of crack formation is considered to be partly due to water favouring stress corrosion at the crack tips and partly due to the internal stresses induced by changing the rock humidity. The result is an increased short- and long-term deformability and a reduced long-term strength /Lajtai et al. 1987/.

The effect of temporary internal confinement increases the rock strength. This effect, known as 'dilatancy hardening', is related to the possibility of pore water drainage at a given applied strain rate /Brace and Martin 1968/.

3.5 Conclusions

The following conclusions can be drawn:

- For the conditions relevant to repository performance, the intact rock part of the repository host rock is in the brittle regime because 1) temperatures will not be remotely close to melting temperatures and 2) confining stresses will be well below the uniaxial compressive strength, which is usually thought to be a measure of the brittle to semi-brittle transition pressure /Ladanyi 1993/. This means that time dependent intact rock behaviour is a question of fracture mechanics and crack growth. It follows from this that fracture mechanics concepts, like the crack initiation stress and the crack damage stress, are relevant for the description of at least some aspects of creep in intact rock /Martin 1997/.
- There are threshold stress levels below which no creep movements in the intact rock take place. This is a consequence of the intact rock being in the brittle regime, obeying laws of fracture mechanics. No creep movements can take place below the crack initiation stress, which may correspond to about 40%–60% of the laboratory determined strength of crystalline rocks /Martin 1997, Damjanac and Fairhurst 2010/. The crack initiation stress level can therefore be regarded as a measure of the creep threshold stress in intact rock.
- If the stress level exceeds the creep limit (crack initiation stress) such that creep movements are initiated but not sufficiently high to exceed the crack damage level, which may correspond to about 80% of the laboratory determined strength of crystalline rocks /Martin 1997/, then the creep movements are likely to die out fast if the porosity and the density of initial micro cracks are low. This is in agreement with observations of strongly retarded creep in intact crystalline rock and with the log-time creep law /Pusch and Hökmark 1992/.
- The dependence on deviator stress and confining stress is in good agreement with the notion that the coefficient A in the log-time creep law (Equation 1-3) is a measure of the stress state.

The conclusions above are largely qualitative in nature, and do not give any direct indications of the magnitude of possible repository near-field rock deformations caused by intact rock creep that could be expected during the 100,000 years following deposition and closure. To make quantitative estimates, one is still reduced to the uncertain use of extrapolation of results obtained from short-term tests. Examples of such tests are the uniaxial loading tests referred to in /Pusch and Hökmark 1992/ (c.f. Figure 3-1). These extrapolations point towards axial creep strain values of between 0.1% and 0.3%. These values are of the same order of magnitude as the axial elastic strain for uniaxial loads below or around the failure load for granitic rocks /Martin 1997, Hakala 1996/. This means that the main conclusion here should be that the effects of intact rock creep cannot be of concern for the geometry of tunnels and deposition holes. If time dependent rock mass behaviour is to be a problem, it must be attributed to movements along rock discontinuities.

The log-time creep laws upon which the extrapolations are based may not be fully valid for thousands of years, considering that the observation periods are some 10 days at most. This may reduce the relevance of the main conclusion. However, the notion of decreasing strain rates is consistent with the findings in the literature and with the fracture mechanics description. In addition, the extrapolated creep strain range, 0.1% to 0.3%, was derived assuming constant loads, while in reality the load on intact rock around repository openings is likely to reduce with time.

4 Creep in discontinuities

4.1 Introduction

The creep displacement in a discontinuity is a function of both the normal stress (σ_n) and the shear stress (τ_p) of the fracture. Under a constant normal stress, the creep deformation is expected to increase as the shear stress is increased and, for a constant shear stress, decrease if the normal stress is increased /Amadei and Curran 1982/.

The long-term shear behaviour of a discontinuity at a constant normal stress is shown schematically in Figure 4-1. The shear model suggests that creep displacements leading to failure may occur for any shear stress level between the peak strength and the residual strength. For a constant applied shear stress, the distance AB represents the creep deformation necessary for failure. If the peak and residual shear strengths and the stiffnesses k_1 and k_2 are constant, then the joint creep deformation necessary to cause failure will be independent of time. However, observed time-dependency of the static coefficient implies that peak shear strength is a function of time /Dieterich 1972, Lajtai and Gadi 1989, Lajtai 1991/.

The creep movement in a discontinuity seems to be of a different nature depending on whether the fracture is planar or rough, and on whether it is clean or filled. Creep in planar joints is presumed to be controlled mainly by an adhesion-frictional mechanism /Bowden and Curran 1984/.

In an unfilled rough discontinuity, the process of time-dependent shearing of individual asperities may explain creep. The time element in this process is due to stress concentrations at asperities along the joint surface that cause slip as the asperities yield progressively and the shear stresses are redistributed to other intact asperities /Schwartz and Kolluru 1982, Ladanyi 1993/.

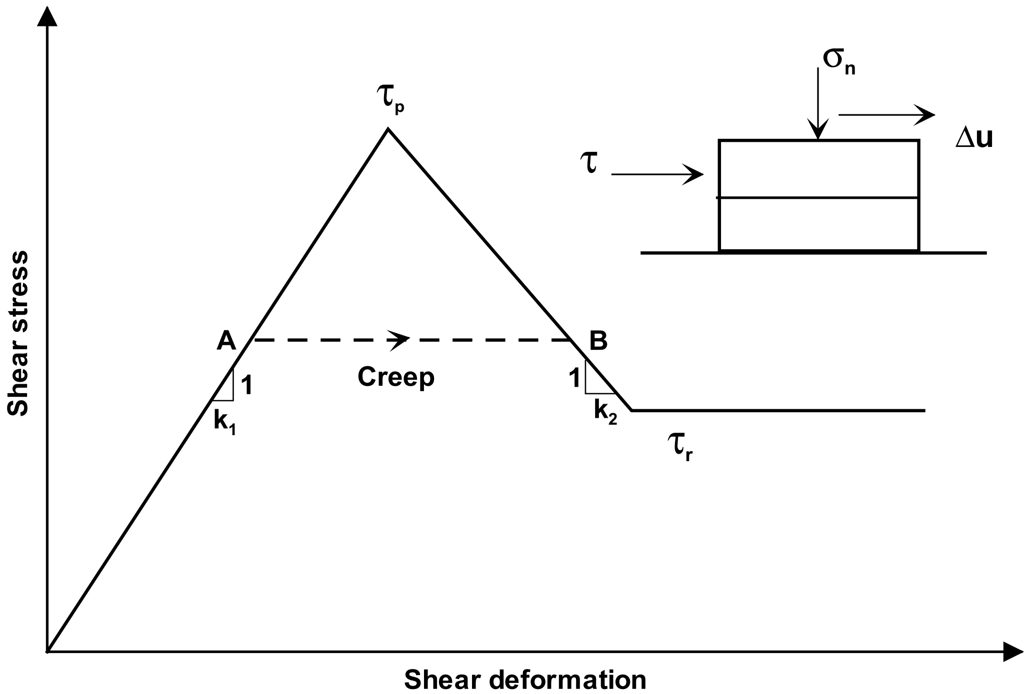

Figure 4-1. *Hypothetical stress-deformation path of a discontinuity at a constant normal stress (After /Amadei and Curran 1982/).*

In a filled discontinuity, the creep is influenced both by the filler shear strength and the shear strength of the filler-rock interface. The creep rate is dependent on the ratio between the filling thickness and the amplitude of undulation. If the particle size is fine and the filling thickness large enough, the roughness of the discontinuity becomes ineffective. This may produce conditions similar to those of a smooth discontinuity, where creep is controlled mainly by the characteristics of the filling material /Höwing and Kutter 1985, Malan et al. 1998/.

Factors controlling the creep movements of discontinuities as found in the literature are the main topic in this chapter. However, since fracture creep movements are related to the fracture shear strength, the starting point is a brief review of factors that control the shear strength in discontinuities of different kinds (4.2). Included in this section are also some results of time-dependent shear strength (4.2.4). In the main section (4.3) results from creep tests of unfilled joints (4.3.1) and filled joints (4.3.2) are presented. Factors that influence the creep rate, such as stress state, joint surface strength and surface roughness, filling thickness and particle size distribution, are also described under section (4.3). Finally, the chapter is closed with some conclusions (4.4).

4.2 Fracture shear strength

4.2.1 General

Shearing along a natural unfilled discontinuity is resisted by friction caused by over-riding or fracturing of microscopic asperities. Larger scales may also involve undulation and rock bridges. Smooth and rough joint walls will usually result in widely different shear strength and deformation characteristics.

The original theory of friction assumes that the material on either side of the contact surface is rigid. The gouge that accumulates on a sliding surface, however, is evidence of intense deformation and the initial surface roughness changes rapidly as the surface wears /Lajtai and Gadi 1989/. Under the influence of a steadily increased shearing force and a constant normal load, the mobilized friction resistance depends on:

1. the initial condition of the surface,
2. wear of the surface through displacements,
3. time and velocity related effects.

The interpretation of the shear strength usually involves some variation of Mohr-Coulomb's linear equation, in which the peak shear strength is given by the following expression:

$$\tau_p = c + \sigma_n \tan \phi_p \qquad (4\text{-}1)$$

where c is the joint cohesive strength, σ_n is the effective normal stress and ϕ_p the peak friction angle. The expression can also be used for the residual shear strength by taking $c = 0$ and substituting the peak friction angle by the residual friction angle ϕ_r , see Figure 4-2.

The peak shear strength envelopes for rock joints are in reality not linear but strongly curved. Both c and ϕ are stress dependent variables and also scale dependent /Barton and Choubey 1977/.

4.2.2 Unfilled discontinuities

Shear strength of planar joints

The shear strength of two flat rock samples in contact is a result of frictional forces between contact points of mineral grains in two rock surfaces. Different geological minerals have different coefficients of friction. The measured friction between two rock samples is an average of the friction forces of the grains composing the rock sample.

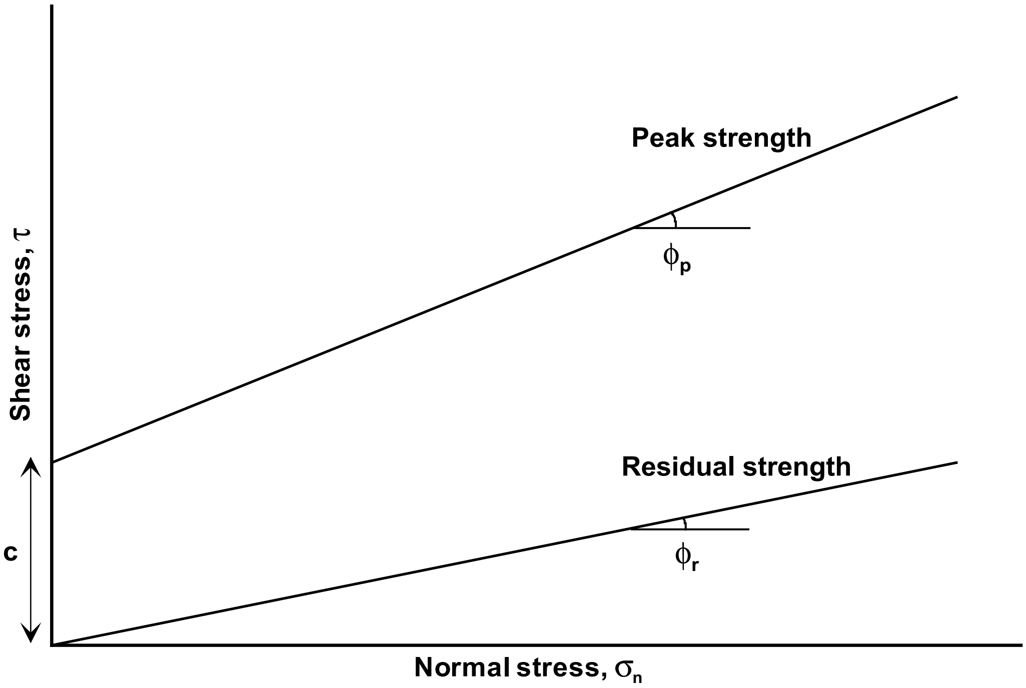

Figure 4-2. *Shear stress versus normal stress for a schematic direct shear test.*

The shear strength of two smooth planar surfaces in fresh rock is usually described by the following equation /Barton 1974/:

$$\tau = \sigma_n \tan \phi_b \tag{4-2}$$

where ϕ_b = basic friction angle. The basic friction angle represents the minimum shear resistance of a smooth planar surface in fresh rock, and is considered to be a material constant dependent on the mineralogy and the moisture of the rock. The parameter is usually approximately equal to the residual friction angle (ϕ_r). However, the residual friction angle can be lower than the basic friction angle due to weathering or alteration effects.

The basic friction angle for some typical rock types listed by /Barton and Choubey 1977/ is presented in Table 4-1. The values indicate that a basic friction angle in the interval 25°–35° can be used as a reference value when estimating the shear strength of unfilled joints in common unweathered rocks. The basic friction angle for samples of granite from Äspö Hard Rock Laboratory has been evaluated to 28.7° ±2.7° /Olsson 1998/.

Table 4-1. Basic friction angles of various unweathered rocks obtained from flat surfaces. (Modified after /Barton and Choubey 1977/)

Rock type	Moisture conditions	Basic friction angle, ϕ_b (deg)
Basalt	Dry	35–38
	Wet	31–36
Fine-grained granite	Dry	31–35
	Wet	29–31
Coarse-grained granite	Dry	31–35
	Wet	31–33
Gneiss	Dry	26–29
	Wet	23–26

Shear strength of rough joints

A natural discontinuity surface in hard rock usually includes undulation and asperities. The joint surface roughness causes a displacement to occur in the normal direction, i.e. perpendicular to the shear direction. The behaviour can be represented by a dilatation angle (*i*). The degree to which a rock joint dilates when sheared has a significant influence on the joint shear strength.

At low normal stresses, where rough joints suffer insignificant or relatively little damage during shear, the following equation may be used as a first approximation to the peak shear strength /Patton 1966/:

$$\tau_p = \sigma_n \tan \phi_b + i \qquad (4\text{-}3)$$

where σ_n is effective normal stress, ϕ_b the basic friction angle and i the peak dilation angle. Equation 4-3 is not valid at high normal stresses, when the strength of the asperities will be exceeded and the teeth will tend to break off, resulting in a shear strength behaviour closely related to the intact material strength /Hoek et al. 1994/.

According to /Barton 1971/ the peak dilation angle at low normal stress is related to the applied normal stress (σ_n) and the uniaxial strength (σ_c) as:

$$i = 10 \log_{10} (\sigma_c / \sigma_n) \qquad (4\text{-}4)$$

Using Equation 4-4 for a discontinuity in granite, under an applied normal stress in the interval 2–4 MPa with a uniaxial stress of σ_c = 150 MPa, gives a peak dilation angle estimate in the interval of 16–19°. The mean peak dilation angle from test samples of joints in granite from Äspö Hard Rock Laboratory, under a similar normal stress, has been evaluated to 13.8° ±3.2° /Olsson 1998/.

4.2.3 Filled discontinuities

Rock joints may naturally be filled with frictional or cohesive material. The filling material is rarely of a better quality than that of the rock blocks. In general, filling materials are of poor quality: sand, silt or clay.

Filling materials can be classified with respect to the way of formation:

1. sedentary filling (stationary) – originating through local alteration or weathering along the discontinuity. Stationary, formed at the location,
2. sedimentary filling (transported) – originating through soils washed in by circulating water,
3. gouge filling- originating through soils generated by shear movement.

The shear strength can be reduced drastically when part or the entire surface is covered by soft filling material. The most obvious effect of a filling material is to separate the joint surfaces and thereby reduce the contact between opposing asperities. However, the shear strength will also be influenced by the surface texture, the nature of the filling material and the characteristics of the joint-fill interface /Papaliangas et al. 1990/.

The shear strength decreases with increasing filling thickness /Papaliangas et al. 1990, Pereira 1990/. For planar surfaces a thin clay coating will result in a significant shear strength reduction. For a rough or undulating joint, the filling thickness has to be approximately twice the amplitude of the undulation before the shear strength is reduced to that of the filling material /Phien-wej et al. 1990/.

/Barton 1974/ has presented a comprehensive review of the shear strength of filled discontinuities. Friction angles of typical filling minerals can also be found in the reference work Landolt-Börnstein /Rummel 1982/. A summary of the shear strength of typical discontinuity fillings is presented in Table 4-2 and Table 4-3. These values can be used for estimating reference values of shear strength for a coated or filled joint.

Table 4-2. Shear strength of filled discontinuities and filling material (Modified after /Barton 1974/).

Rock type	Description	Cohesion, *c* (MPa)	Friction angle, ϕ_p (deg)
Basalt	Clayey basaltic breccia	0.24	42
Diorite and porphyry	Clay gouge (2% clay, PI=17%)	0	26.5
Granite	Clay filled faults	0–0.1	24–45
Granite	Sandy loam fault filling	0.05	40
Schists and quartzites	Stratification with thin clay	0.61–0.74	41
Schists and quartzites	Stratification with thick clay	0.38	31

Table 4-3. Friction angle of mineral as a function of dry and wet surface (Modified after /Rummel 1982/).

Rock type	Surface condition	Friction angle, ϕ_p (deg)
Biotite , Canada	cleavage, dry	17.0
	cleavage, wet	7.4
Chlorite, Vermont, USA	polished, dry	28.0
	polished, wet	12.0
Muscovite	cleavage, dry	23.0
	cleavage, wet	13.0

4.2.4 Time-dependent shear strength

The friction angle of a joint surface is not a constant, but a variable that depends on time-related parameters such as displacement rate, accumulated displacement and the duration of stationary contact. Any process that increases the 'true area of contact' in the discontinuity will increase the frictional resistance of the joint /Lajtai and Gadi 1989/.

Displacement-strengthening involves change of the sliding surface into one that is in equilibrium with the imposed velocity. Once such a surface is created, the angle of friction will no longer change with further displacement. The displacement required to reach equilibrium conditions can, however, be quite large. Shear tests performed by /Lajtai and Gadi 1989/ on granite blocks with initially smooth and polished surfaces, gave friction angle increases of approximately 14 degrees for an accumulated displacement of 500 mm, see Figure 4-3.

For clean, rough joint surfaces, the friction coefficient is independent of the duration of stationary contact, while for joints in which some gouge has accumulated it increases with the contact duration /Dieterich 1972/. One explanation put forward for the observed increase in frictional resistance, during stationary contact, is that it is attributed to time-dependent compaction and subsequent strength increase of the gouge material accumulated in the discontinuity. Experiments by /Dieterich

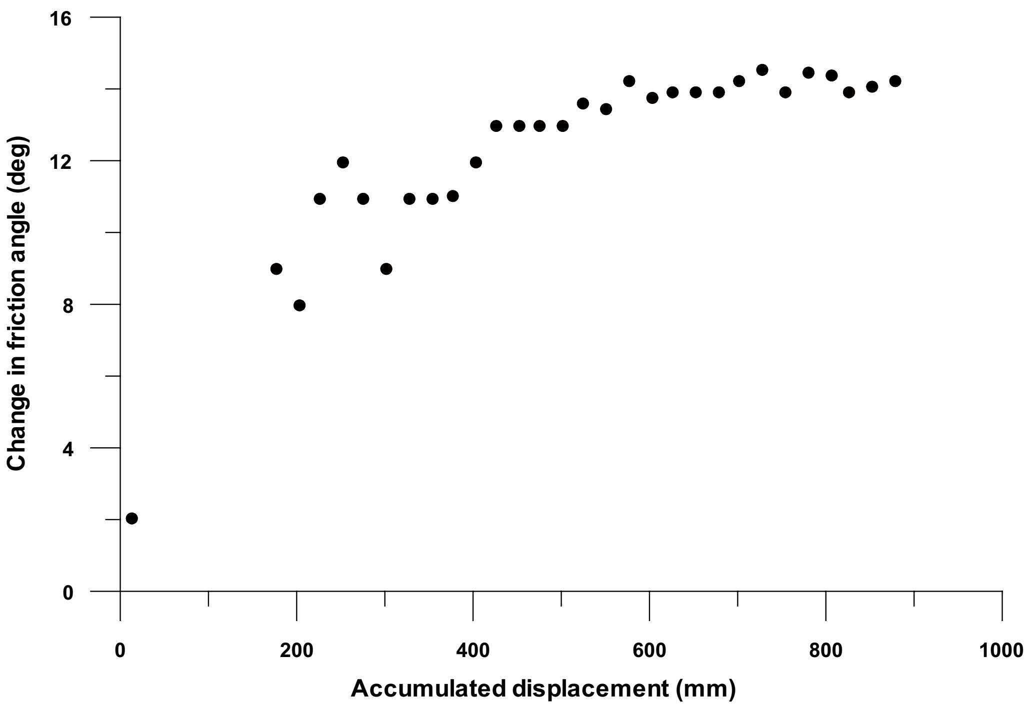

Figure 4-3. *The increase in friction angle for a joint in granite with wear of the surface (After /Lajtai 1991/).*

1972/ on rough as well as smooth joint surfaces showed that the time-dependence of the static coefficient of friction could be expressed as:

$\mu(t) = \mu_o + A\log(t + 1)$ (4-5)

where μ_o is the initial coefficient of friction, A is a constant and t is the duration of contact in seconds. Values for μ_o and A for granite and quartzite are given in Table 4-4. The increase of μ with time appears to be independent of normal stress. Using the equation for a discontinuity in granite under 2 days stationary contact will result in an increase of 5–6 degrees in the friction angle. This is in general agreement with results by /Lajtai and Gadi 1998/ that show an increase of 3–4 degrees during a shear loading delay of 2–4 days, see Figure 4-4.

The test results presented above indicate that, in rock joints under shear conditions that accumulate gouge fill, the creep is most likely transient. The reason is that the time-dependence of the static coefficient of friction implies that the peak shear strength will increase with time. This will give a reduction in the ratio of the applied shear stress to the peak shear strength, which is found to control the magnitude of creep displacements /Amadei and Curran 1982/. The described state will, however, not arise if the characteristics of a joint are a product of earlier shearing under a similar loading condition since wear and compaction of the accumulated filling will not occur in these circumstances.

It should be noted that the time strengthening achieved through a delay in loading is temporary. Once the friction resistance is overcome and displacement resumes its pre-delay rate, the friction resistance drops back to the level established before the loading delay /Lajtai and Gadi 1989/.

Table 4-4. Values for coefficients μ_o and *A* for granite and quartzite at a normal stress of approximately 10 MPa (After /Dieterich 1972/)

Rock type	Coefficient of static friction, μ_o	Constant, A
Sandstone	0.73	0.015
Granite	0.79	0.022
Quartzite	0.84	0.020
Greywacke	0.83	0.012

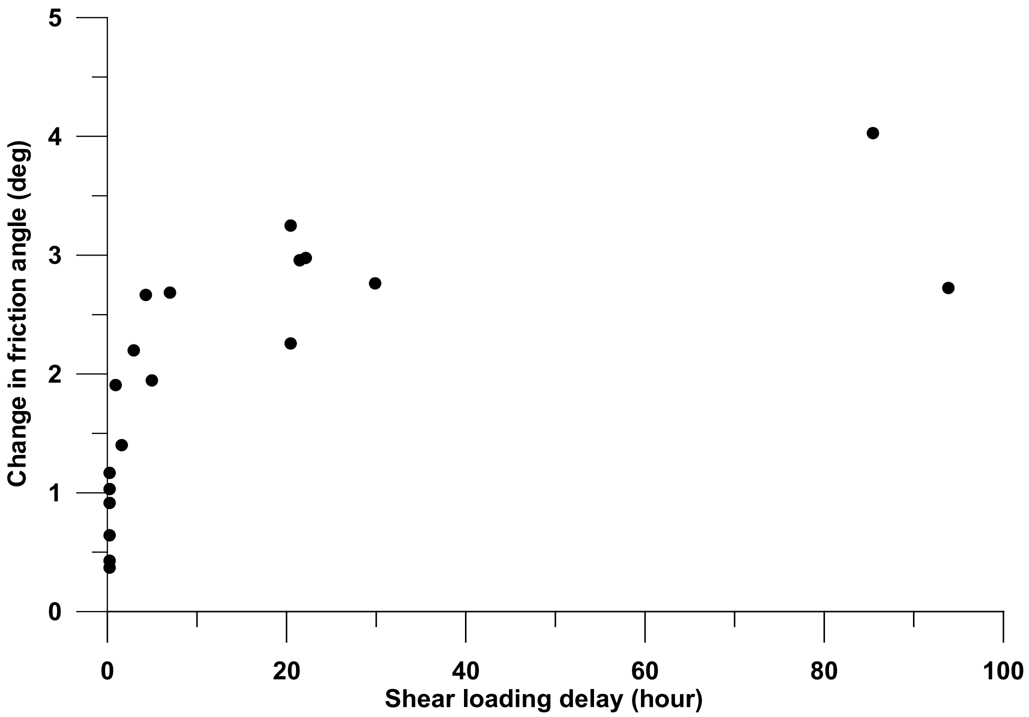

Figure 4-4. *The increase in the friction angle following stationary contact in a direct shear test. (After /Lajtai 1991/)*

4.3 Results from creep tests

4.3.1 Unfilled discontinuities

The typical creep behaviour of an unfilled discontinuity is presented in Figure 4-5. The test was conducted on a sample of water-saturated Westerly granite containing one joint inclined at 30° relative to the axial compression /Wawersik 1974/.

The results show an instantaneous elastic strain followed by a transient and steady-state creep. The instantaneous joint deformations exceed the time-dependent response by several orders of magnitude. The steady-state creep is in the actual experiment just a few percents of the total strain.

Another example of creep tests performed on an unfilled joint is shown in Figure 4-6. The examined fracture is an excavation-induced rough discontinuity in porphyry lava with very high uniaxial compressive strength (435 MPa). The fracture was closely matched and not subjected to any previous shear movements. The normal load was 1.0 MPa /Malan et al. 1989/.

The sample failed at a shear stress of 1.7 MPa giving an apparent friction angle of 59.5°. The total displacement before failure was only about 10 µm. Unexpected trends of creep in the steady-state creep phase of the first two loading stages are explained by /Malan et al. 1998/ by an effect of slight changes in temperature (±1°C) that have an influence on the instrumentation.

Several researchers report that the creep behaviour of unfilled discontinuities has the same general character as that of intact rock, but with a strain magnitude that is larger /Amadei and Curran 1982, Schwartz and Kolluru 1981, 1982, Wawersik 1974/. An example of these types of results is presented in Figure 4-7. The creep curves in the figure are from four different configurations: intact rock, a cohesive joint oriented at θ=30°, a cohesive joint oriented at θ=60°, and a cohesion less joint oriented at θ=30°. All four tests were conducted at a stress magnitude of 0.4 σ_c. The parameter θ is the angle between the normal to the joint plane and the direction of the applied stress.

All curves exhibit a characteristic non-linear primary creep followed by secondary steady-state creep. In all samples including a joint, the creep strains were higher but not very much higher, than for the intact sample. Increasing θ from 30° to 60° or decreasing the cohesion to zero also lead to higher creep /Schwartz and Kolluru 1981/.

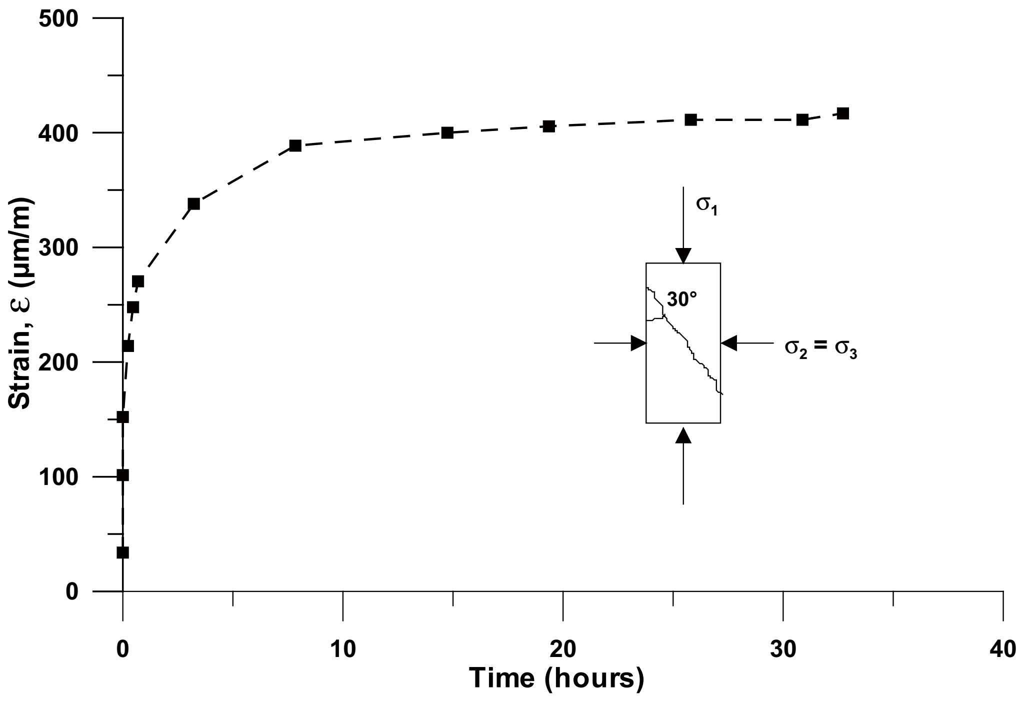

Figure 4-5. *Axial strain versus time for a joint in water-saturated granite under constant principal stress difference (σ_1–σ_3 = 60.5 MPa) and confining pressure (σ_3 = 20.7 MPa). Note that the creep rate after 30 hours is very small. The slope of the curve gives an approximate value of $1 \cdot 10^{-11}$ s^{-1}. (After /Wawersik 1974/)*

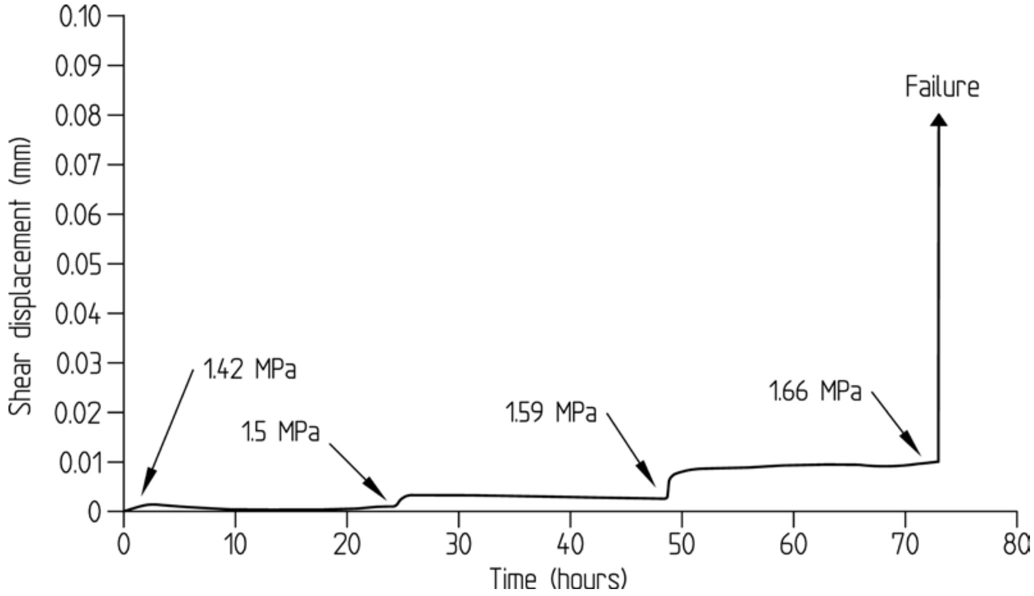

***Figure 4-6.** Shear creep behaviour of an excavation-induced tensile fracture in porphyry lava with a stepwise increase in shear stress. The normal stress was 1 MPa. The sample was tested at the ambient humidity of 50%. (After /Malan et al. 1998/)*

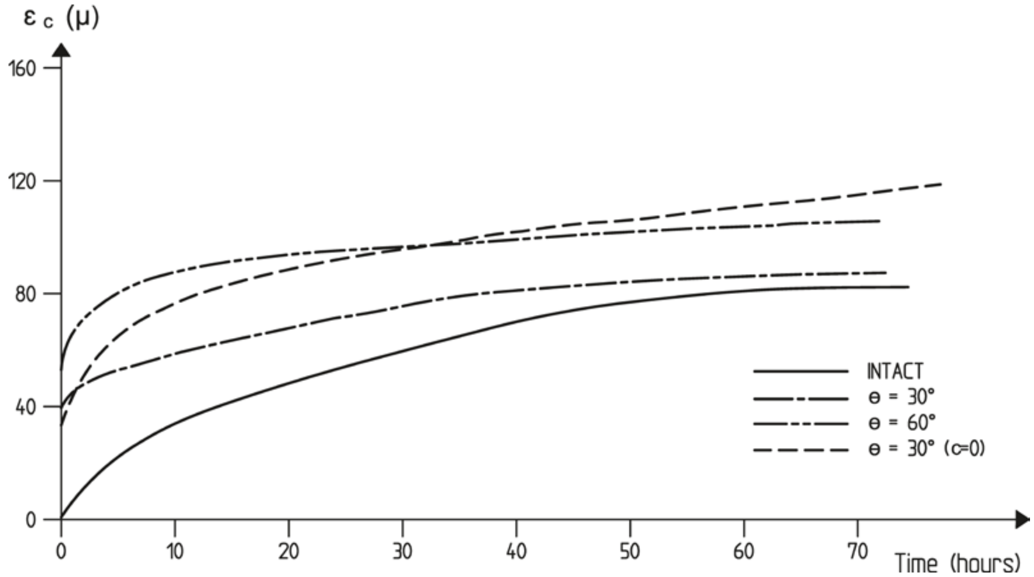

***Figure 4-7.** Creep curves for intact and jointed samples at 0.4 σ_c stress level. The parameter θ represents the angle between the direction of the applied axial stress and the direction of the normal to the joint plane. (After /Schwartz and Kolluru 1981/).*

A possible explanation for the agreement in creep behaviour between intact rock and unfilled discontinuities could be a similarity in the creep process. The observed state with decreasing time-dependent displacement can in both cases be explained by stress concentrators that are exhausted by micro fracturing of the rock material.

Stress dependence

Several studies indicate that significant creep in fractures will not occur unless the shear stress magnitude is above some threshold value /Amadei and Curran 1982, Bowden and Curran 1984, Schwartz and Kolluru 1982/.

The results from a test of marble, performed by /Amadei and Curran 1982/ show a limit of the shear stress/strength ratio (τ/τ_p) of approximately 0.5, beyond which there is a plain increase in the creep movement along the discontinuity. For low values of the shear/strength ratio, creep displacement of the intact rock dominates.

Figure 4-8 shows the results of several shear tests performed at shear stress/strength ratios ranging between 0.6 and approximately 1.0 on artificial discontinuities in shale /Bowden and Curran 1984/. The test results display creep rates which are small and decrease rapidly with time for stress/strength ratios less than 0.7, while for ratios larger than 0.9 the creep displacements become significant.

/Amadei 1979/ hypothesized that the ratio of the applied shear stress to the peak shear strength (τ/τ_p) was the critical factor governing discontinuity creep. However, later laboratory investigations have shown that the creep rate is not only a function of shear stress to shear strength ratio, but also of the absolute values of the shear and normal stresses.

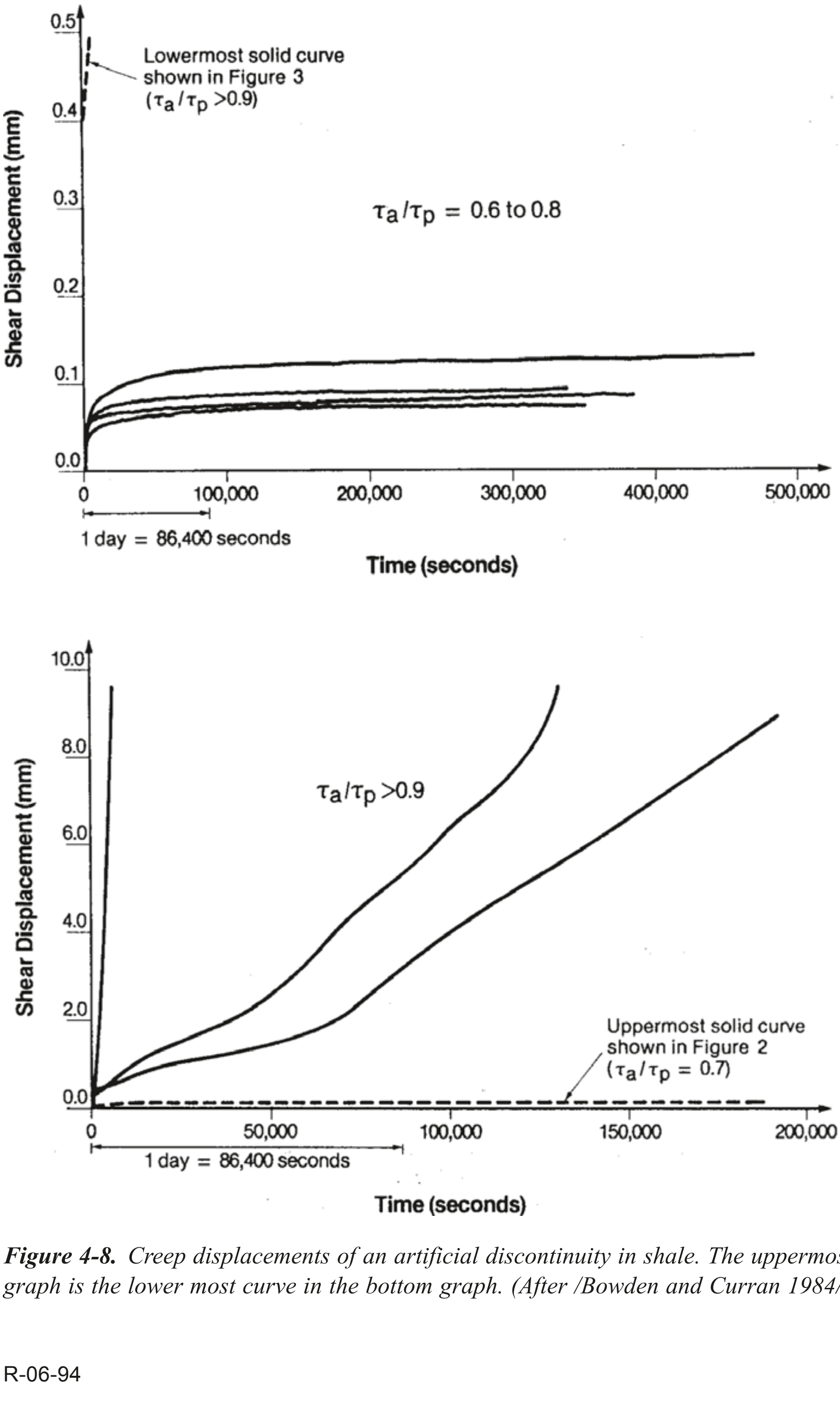

Figure 4-8. *Creep displacements of an artificial discontinuity in shale. The uppermost curve in the top graph is the lower most curve in the bottom graph. (After /Bowden and Curran 1984/)*

__Influence of joint surface condition__

Condition of the joint surface such as roughness, wall strength, moisture content and weathering, have an influence on the peak shear strength of the discontinuity. By a change in the degree of interlocking or a change of the strength of individual joint asperities, the ratio of the applied shear stress to shear strength of the joint is changed, which in turn influences the creep rate. Results from creep tests that in any real sense demonstrate the influence of the surface condition of unfilled joints have not been found. This is probably due to difficulties in reproducing identical surface conditions from one test to another.

4.3.2 Filled discontinuities

The creep behaviour of a discontinuity with gouge filling in a shear test performed by /Malan et al. 1998/ is shown in Figure 4-9. The shear stress was increased stepwise and both primary and secondary creep can be observed. The sample failed at a shear stress of about 0.5 MPa giving a friction angle of approximately 26.5 degrees. The total shear displacement before failure amounts to about 1.1 mm. The normal displacement recorded in the test indicated compaction of the gouge layer with increasing shear stress, despite the constant normal stress.

Opinion is divided about the number of creep stages that can be identified in the case of filled discontinuities. According to test results by /Höwing and Kutter 1985/, no secondary creep stage is found for filled rock discontinuities, i.e. no transition phase with constant creep velocity between primary and tertiary creep was observed. /Malan et al. 1998/, on the other hand, focus on steady-state creep in their presentation of results. The reason for the divergence is probably differences in the test conditions, i.e. initial conditions, the stress and displacement magnitude, the duration of the tests and the grain size distribution of the filling material.

The factors of importance for the creep rate of filled discontinuities seem to be:

1. shear stress/shear strength ratio,
2. absolute stress magnitudes,
3. gouge thickness,
4. grain size distribution.

The influence of these factors on the creep behaviour is discussed below.

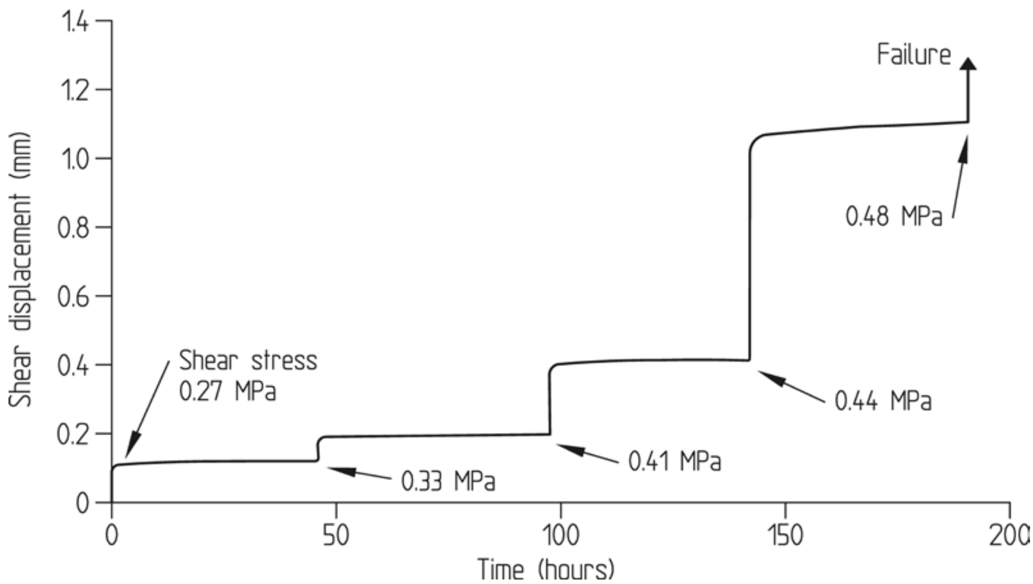

Figure 4-9. *Shear creep behaviour of a discontinuity with bedding plane gouge -collected from Hartebeest-fontein mine. The normal stress was 1 MPa. The gouge thickness was 2 mm and the humidity 50%. (After /Malan et al. 1998/)*

Stress dependence

/Malan et al. 1998/ investigated the effect of the magnitude of the shear stress and normal stress on a gouge filled fracture. The tests were conducted by increasing the shear load, in a stepwise fashion, under a normal stress at three different magnitudes: 0.5, 1.0 and 1.5 MPa. In Figure 4-10 incremental shear displacements for different shear load increments under a normal stress of 0.5 MPa are illustrated. The magnitude of instantaneous response was very prominent in the test and increased with the shear stress to shear strength ratio (τ/τ_p).

The test also showed a steady-state creep rate that increased with the shear stress magnitude, see Figure 4-11. The behaviour in tests with normal loads of 1.0 MPa and 1.5 MPa was similar. It was, however, observed that the magnitudes of the primary and steady-state creep rate are not only a function of the shear stress to shear strength ration (τ/τ_p), but also of the absolute values of the shear and normal stresses. For a 1.5 MPa normal load, the steady-state rate at a (τ/τ_p) ratio of 0.93 was more than 3 times larger than that for tests with a 0.5 MPa normal load.

/Malan et al. 1998/ suggest the following relation between the steady state creep rate and the stress/strength ratio:

$$\dot{D}_{ss} = A\left(\frac{\tau}{\tau_s}\right)^n \tag{4-6}$$

where *A* and *n* are parameters that depend on the thickness and type of filling, in addition to the absolute stress level.

Influence of filling thickness

The effect of the filling thickness of the shear creep in a fracture has been investigated by /Höwing and Kutter 1985/ and /Malan et al. 1998/. Figure 4-12 presents the results by /Höwing and Kutter 1985/ with kaolin filling and Figure 4-13 the results by /Malan et al. 1998/ with medium to fine grained sand filling.

The test results clearly illustrate that creep is dependent of the filling thickness. Both initial creep velocity and steady-state creep rate increase with the filling thickness.

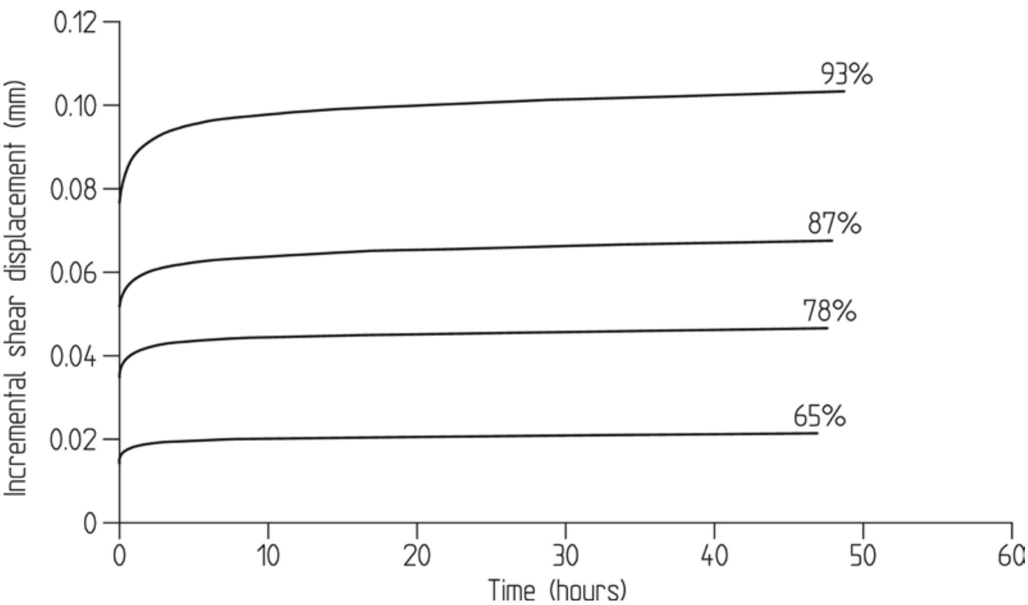

Figure 4-10. *The effect of shear stress magnitude on the creep behaviour. The percentages indicate the magnitude of the shear stress relative to the shear strength (0.28 MPa). The normal stress was 0.5 MPa and the gouge thickness 2 mm. (After /Malan et al. 1998/)*

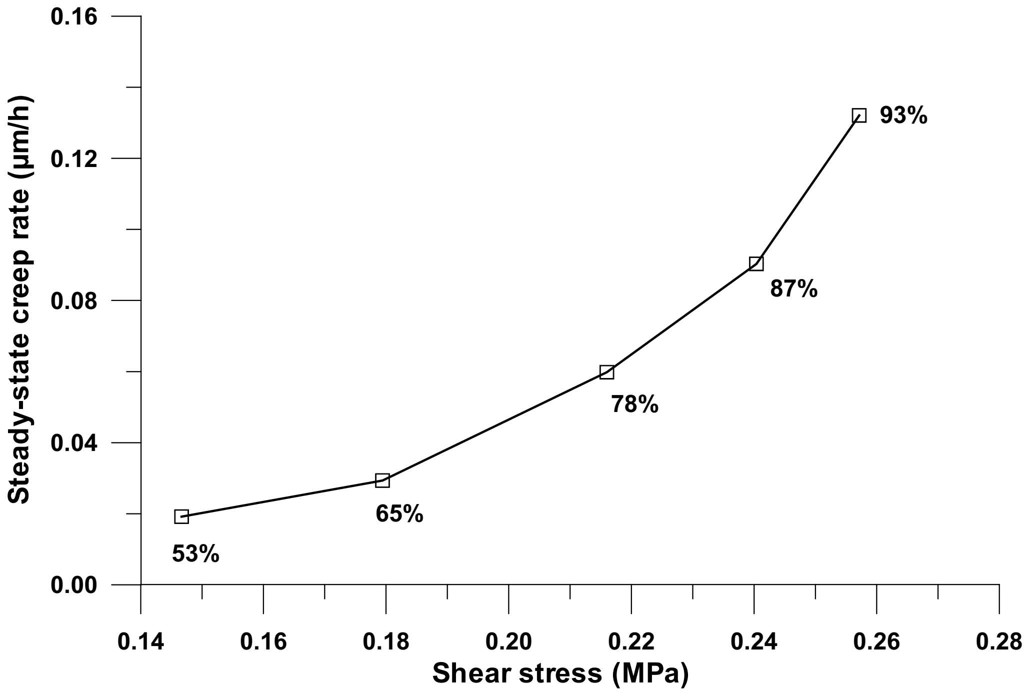

Figure 4-11. *The effect of shear stress magnitude on the creep behaviour of a gouge filled fracture. The percentages indicate the magnitude of shear stress relative to the shear strength (0.28 MPa). The normal stress was 0.5 MPa and the gouge thickness 2 mm. (After /Malan et al. 1998/)*

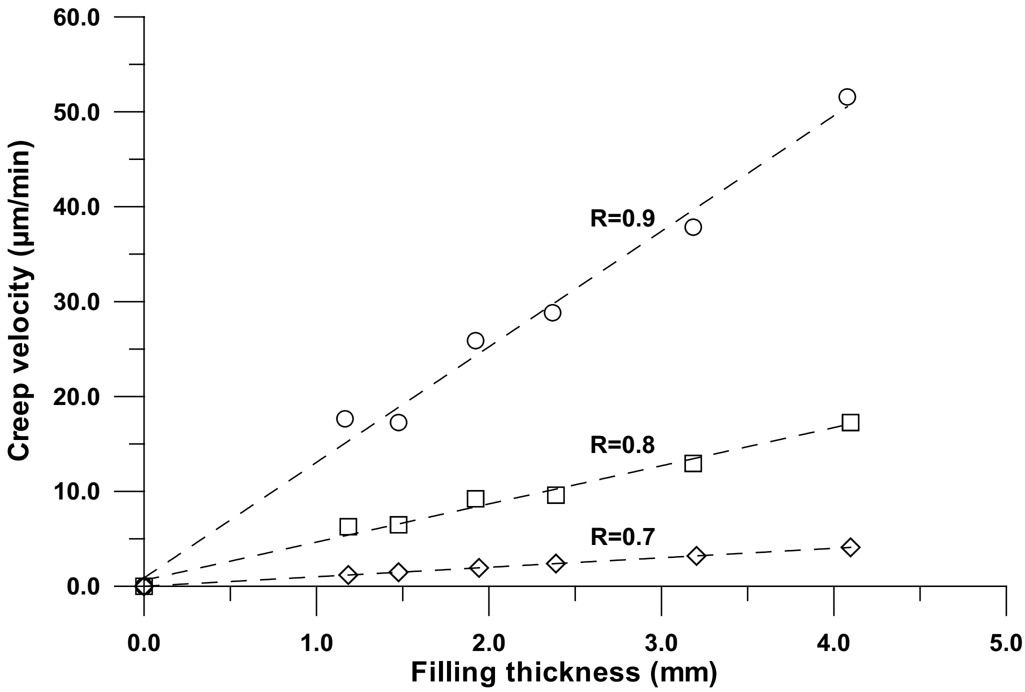

Figure 4-12. *Initial creep velocity (v after 1 min) as a function of filling thickness. The filling material was Kaolin with a clay content of 0.75 and a consistency index of 0.75. The normal stress was 1 MPa and the ratio ($R = \tau/\tau_R$) was varied between 0.7 and 0.9. (After /Höwing and Kutter 1985/)*

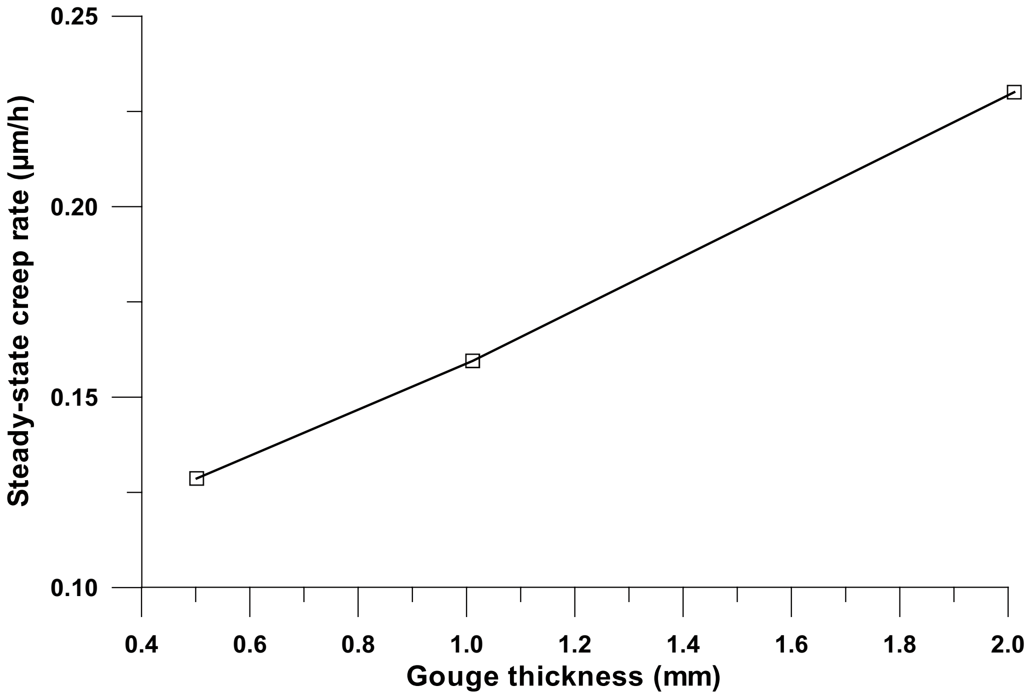

Figure 4-13. *Steady-state creep rate as a function of filling thickness. The filling material was a medium to fine sand. The (τ/τ_p) ratio was 0.86 for the 0.5 mm thickness, while 0.87 for the 1 mm and 2 mm thickness. The samples were tested at 50% humidity. (After /Malan et al. 1998/)*

The results are most likely dependent upon the asperity amplitude of the fracture surface in relation to the grain size of the infilling. The reason for increased creep velocity with thicker infilling, is probably that the interlocking effects of opposing asperities are reduced. This means that the creep behaviour of the fracture approaches the creep behaviour of the filling material (See /Phien-wej 1990, Pereira 1990 Papliangas et al. 1990/).

Influence of particle size distribution

According to results by /Höwing and Kutter 1985/ the creep rate in the primary and tertiary phase is directly dependent on the clay content of the infilling. Figure 4-14 presents the initial creep velocity in a triangular concentration chart for grain-size fractions. Lines of equal creep velocity run parallel to lines of equal clay content in the diagrams. The creep rate of soils with coarse grains, i.e. with a clay content between 0.1–0.3, is rather small. The /Höwing and Kutter 1985/ explanation for the observed results is that with increasing clay content the coarse grains are more and more enclosed by clay particles. The friction between the coarse particles is therefore increasingly reduced.

The increase of the creep velocity in the primary phase (v_1) with the clay content can be expressed by an exponential function /Höwing and Kutter 1985/:

$$v_I = a \cdot e^{bT} \tag{4-7}$$

where, a and b are coefficients that are determined empirically and T is the clay content.

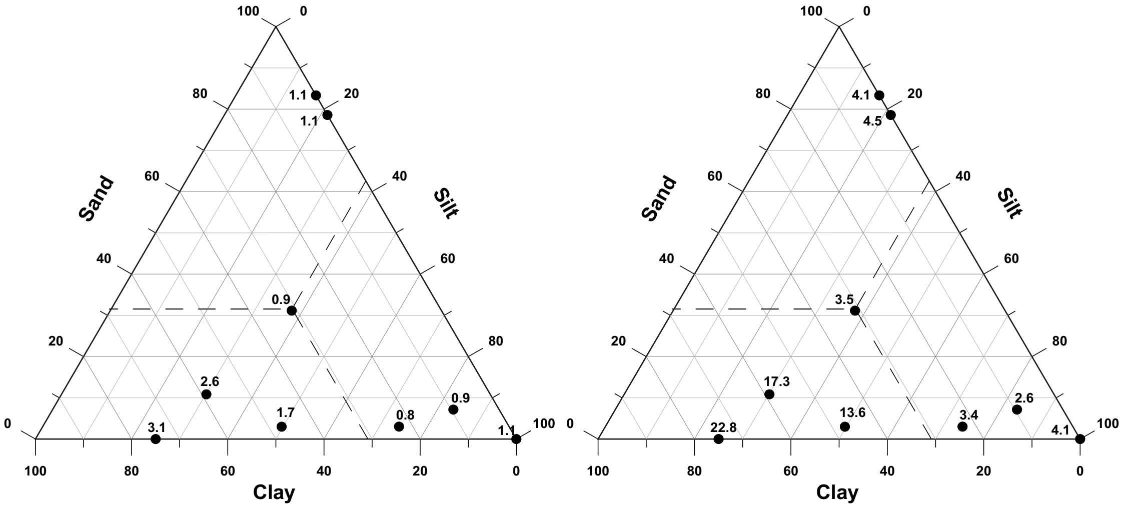

Figure 4-14. *Initial creep velocity (v after 1 min) as a function of particle size distribution of the filling material. The filling thickness was 2 mm and the consistency index 0.75. The normal stress was 1 MPa. The diagram on the left shows a shear stress ratio R = 0.7 and the diagram on the right a ratio R = 0.9. (After /Höwing and Kutter 1985/).*

4.4 Conclusions

One general finding of the literature study is that results from creep laboratory tests on natural discontinuities in granite are sparse. In addition, experimental data for all types of rock involve short times and loading conditions that are different from those that will apply in the repository. Therefore, it is not meaningful to try and conclude anything about the nuances of time-dependent fracture behaviour. Instead it is necessary to capture the essence of the findings with regard to the objectives of this study, i.e. to find if there is any support of the assumption that threshold values exist, such that creep along fractures does not take place unless the stress/strength ratio exceeds that threshold. From this standpoint the following conclusions are drawn:

- The creep rate is not only a function of the shear stress to shear strength ratio, but also of the absolute values of the shear and normal stresses. Results by /Malan et al. 1998/ show a steady-state rate more than 3 times larger for a test at 1.5 MPa normal load than for those at 0.5 MPa at a stress/strength ratio of 0.93. This suggests that only laboratory tests performed at stress magnitudes that are similar to those that occur in the real rock mass are relevant to the forecast of creep movements.
- It is relevant to distinguish between filled and unfilled discontinuities. Typical test results reported by /Malan et al. 1998/, for instance, show approximately 100 times larger total displacements in gouge-filled discontinuities than in rough unfilled joints. This general observation applies in particular for clay-filled joints.
- For unfilled joints the tendency to creep seems to vary from almost none at all to very small:
  - o The fracture tested by /Malan et al. 1998/ shows no creep at all for stress/strength ratios below about 95% (c.f. Figure 4-6).
  - o The results of the triaxial tests conducted by /Schwartz and Kolluru 1981/ indicate that the contribution of the fractures was on the level of that of the intact rock portion of the tested samples (c.f. Figure 4-7).
  - o The triaxial test described by /Waversik1974/ gave a strain rate of about $1 \cdot 10^{-11}$ s^{-1} after about 30 hours. If this figure is used to forecast the future creep rate for that sample using the log-time creep law, then the creep strain would amount to not more than 0.1% after 100,000 years.
  - o The results obtained by /Bowden and Curran 1984/ indicate that stresses below 50% of the shear strength do not give any creep at all.

- For filled joints the situation is more complex since fillings can be of different types and of different thickness. In addition, the effects of the filling material also depend on, for instance, the roughness of the fracture surfaces. Compared to the results found for unfilled discontinuities, creep rates are higher and creep seems to be initiated at lower stress/strength ratios:
  - Many results indicate the occurrence of steady-state creep at modest stresses (c.f. Figures 4-11, 4-12 and 4-13), as opposed to the transient behaviour found in most experiments conducted with unfilled joints.
  - For the 2 mm gouge-filled fracture tested by /Malan et al. 1998/, the steady-state creep rate was 0.02 μm/h for the lowest stress/strength ratio reported (53%) (c.f. Figure 4-11). This corresponds to a movement of 160 mm in 1,000 years, which shows that, if there should exist a threshold ratio below which creep does not need to be considered, this should be set at lower than 50%.
  - If the same experiment is considered as in the previous point, and the creep law proposed by /Malan et al. 1998/ (Equation 4-6) is applied, then a 20% stress/strength ratio would give a creep rate of about 8 mm in 1,000 years and 10% stress/strength ratio 0.8 mm in 1,000 years.
  - The results obtained by /Höwing and Kutter 1985/ also show a very significant reduction in creep rate when the stress/strength ratio is reduced. For a kaolin-filled fracture the rate was reduced by about 90% when the ratio was reduced from 90% to 70% (c.f. Figure 4-12). Extrapolating these results to 20% and 10% by use of the law proposed by /Malan et al. 1998/ (Equation 4-6), gives rates of 20 mm in 1,000 years and 0.3 mm in 1,000 years, respectively.

As a very general and rough summary of the above the following is concluded:

- For unfilled discontinuities a stress/strength threshold ratio of 30% is a reasonable and probably conservative estimate.
- For filled discontinuities a stress/strength threshold ratio of 10% is a reasonable and probably conservative estimate.

These threshold ratios are estimated values of stress/strength values above which transient creep may be expected. Steady-state creep is not expected to be of any importance, since the surveyed results of both clean and filled discontinuities in most cases show a transient behaviour at a stress/strength ratio less than 50% /Amadei and Curran 1982, Bowden and Curran 1984, Malan et al. 1998, Schwartz and Kolluru 1981, Wawersik 1974/. In addition, creep movement in the repository host rock will not take place under constant load conditions. On the contrary, the load will decrease as a result of the creep-induced stress relaxation.

5 Creep region around a repository tunnel

5.1 Introduction

In Chapter 4, stress/strength threshold ratios for unfilled and filled discontinuities were suggested. For discontinuities subjected to lower stresses than those corresponding to the relevant threshold ratio it will be reasonable to ignore creep movements. If that ratio is exceeded, creep movements will continue until the stress acting on the fracture has been relaxed sufficiently that the stress/strength ratio has dropped below the threshold value.

The intended use of the threshold values is in numerical modelling of jointed rock masses, however, not for assessing the actual temporal development, since this would require application of some rheological model, or a number of rheological models. The threshold values can be used only to find the ultimate state of stresses and deformations, i.e. when shear stresses on fractures have been relaxed sufficiently that all stress/strength ratios have decreased below the threshold.

A way of doing this is to use, for instance, the 3DEC code (or for 2D models the UDEC code) (Consulting Group) and calculate the initial elasto-plastic response to excavation, heating etc. This can be done by using typical values of friction and cohesion and then reducing the shear strength in steps until the ratio between stress and initial strength has dropped below the creep threshold. This technique has been applied in a pilot study /Glamheden et al. 2001/ aiming at assessing the importance of the backfill for limiting the long-term convergence of deposition tunnels.

The pilot study illustrated that low (i.e. conservative) thresholds also gave movement along fractures that were not affected by any disturbance. For fractures far away from the excavation, the in situ stresses were sufficient to initiate movement. This indicates that a stress/strength ratio in excess of the threshold should not be regarded as a sufficient condition for creep to take place. Some further disturbance may be necessary to trigger creep movements.

The induced disturbance around an excavated deposition tunnel can be classified into disturbance that reduces the rock mass strength and disturbance that increases the stresses in the rock mass. Examples of factors that reduce the rock mass strength are:

- excavation method damage,
- stress-induced damage,
- alteration of the rock mass.

Examples of factors that increase or change the stresses in the rock mass are:

- stress redistribution (excavation geometry),
- temperature changes (canister released heat),
- pore pressure changes (effective stress in the joint planes).

The disturbance that each of these factors cause will be of different intensity and extension around the tunnel. In a zone close around the tunnel the disturbance from two or several factors will be combined. At least three zones with different degrees of disturbance may be observed around a tunnel:

- the failed zone (spalling zone),
- the excavation damage zone,
- the disturbed zone.

In the failed zone, rock slabs are detached completely from the rock mass as a result of progressive failure (spalling). In the excavation damage zone, the rock has been damaged due to induced macro- and micro fracturing caused by excavation or stress concentrations. In the disturbed zone, the material behaviour of the rock is essentially unchanged, but the stress state, the temperature field, the pore pressure, etc. are disturbed by the opening /Read 1996/.

It is also common to separate the disturbance into near-field and far-field effects. In the near-field, the process of excavation causes damage to the rock, changes its properties and may produce microcracks or new fractures. In the far-field the rock is affected by the redistribution of stresses etc. caused by the voids represented by the drifts. In the far-field new fractures are not generally induced but existing fractures may react by dilating, closing or shearing /Bauer et al. 1996/.

In this chapter, it is attempted to find support for the assumption that a zone around a repository opening can be defined, beyond which no creep will take place, even if the threshold stress/strength ratio is exceeded. The disturbance and the extension of some of the factors listed above will be discussed briefly.

5.2 Excavation damaged zone

The term Excavation Damage Zone (EDZ) is used to describe the region of rock adjacent to an underground excavation that has been significantly damaged due to induced macro- and micro fracturing caused by excavation or stress concentrations /Fairhurst and Damjanac 1996/. The extension of the EDZ around a tunnel for different excavation methods has been the subject of several research projects during the latest ten years. The most well documented investigations of the EDZ, at full scale in hard rock, are probably the Mine-by Experiment at URL in Canada and the ZEDEX project at Äspö HRL in Sweden /Bäckblom and Martin 1999/.

The excavation method in the Mine-by Experiment consisted of a non-explosive excavation method that involved line-drilling and reaming of a series of 1 m deep perimeter holes around the tunnel diameter and then progressively breaking out the interior of the round using hydraulic rock splitters in a series of production holes. The test tunnel for the Mine-by Experiment at URL in Canada was designed as a circular test tunnel with 3.5 m diameter, excavated at a depth of 420 metres. The rock mass is composed of sparsely fractured granite under high in situ stresses, i.e. $\sigma_1 = 60 \pm 3$, $\sigma_2 = 45 \pm 4$ and $\sigma_3 = 11 \pm 4$ MPa /Read 1996/. The test tunnel was orientated parallel to the intermediate principal stress direction to maximise the potential for progressive failure and damage around the tunnel. The results from the Mine-by Experiment showed that the EDZ is limited to a region within about 0.6 radii from the original perimeter of the test tunnel /Bäckblom and Martin 1999/. Within this region, however, the characteristics of the EDZ vary with position around the tunnel, and are controlled or influenced by such things as nature of the stress concentration, and variations in the geology /Read 1996/.

The ZEDEX project (Zone of Excavation Disturbance Experiment) comprised investigation of two drifts using three excavation methods under similar initial conditions. One tunnel was excavated by a tunnel-boring machine and the other by two smooth blasting methods /Davies and Mellor 1996/. The test tunnels of the ZEDEX experiment were designed to be circular with a diameter of 5 m, excavated at a depth of 420 meters. The dominant rock type at Äspö is grey medium-grained diorite with a mean uniaxial compressive strength of 195 MPa and a mean Young's modulus of 69 GPa. The in situ stress field in the area is evaluated to: $\sigma_1 = 32$, $\sigma_2 = 17$ and $\sigma_3 = 10$ MPa. The results from the ZEDEX project showed that excavation-induced fractures could not easily be separated from the natural fractures. Although the TBM tunnel was relatively fractured, EDZ extent was considered to be about 0.20 m or less. In the drill and blast drift the maximum EDZ extent was encountered on the floor (0.8 m) and the minimum extent on the walls (0.3 m). The differences in extent of EDZ between the two blasting methods were not clear /Bauer et al. 1996, Davies and Mellor 1996/.

From the findings presented above it can be concluded that the EDZ appears to be limited to the order of half or less of the opening radius. The EDZ also appears to be less in a TBM tunnel than a conventional D&B tunnel.

5.3 Stress redistribution

When a tunnel is excavated in a rock mass subjected to a natural inherent stress field, the stresses in the vicinity of the tunnel are redistributed. The stresses around the tunnel are dependent on, among other parameters, the magnitude of the in situ stresses and the tunnel shape /Hoek et al. 1994/.

A reasonable estimation of the stress concentration around a deposition tunnel can be obtained by Kirsch's solution /Kirsch 1898/ for a circular hole in a pre-stressed elastic continuum. The results for a tunnel located in the Scandinavian shield, with a stress anisotropy of maximum 3.5:1, indicate that at a distance of 6 tunnel radii from the opening periphery, the disturbance of the in situ stress is less than 5% for all stress components. This distance is presumed to be a conservative outermost limit of the region where the disturbance caused by stress redistribution is sufficient to trigger creep movements along fractures (provided that the threshold stress/strength ratio is exceeded).

5.4 Temperature changes

In the KBS-3 concept, heat-generating nuclear waste canisters are placed in deposition holes, excavated in the floor of horizontal deposition tunnels. The heat released from the canisters creates a time-dependent temperature field in the ground around the deposition tunnels. The large-scale field influences potential water movements in cracks and fissure zones in the rock. It also creates a thermal stress field in the rock around the tunnel. The temperature change is expected to give rise to stress changes that reduce the rock mass strength, close to a canister hole and increase the stresses at a larger distance.

According to analytical calculations reported by /Claesson and Probert 1996/ a maximum temperature change of approximately 35°C occurs after about 100 years, in the centre of a repository of almost 1 km^2, with canisters deposited according to the KBS-3 concept. The temperature decreases quite slowly and takes approximately 10,000 years to revert to the initial level. The range of influence is approximately 500 m vertically and 1,000 m horizontally, taking a 1°C temperature change as a measure of disturbance, and 250 m and 500 m, if the disturbance measure is 10°C. In another study, /Probert and Claesson 1997/calculated the thermo-mechanical stresses induced by the temperature field. The stress increase in the interior of the repository reached a maximum after about 50 years and amounted to about 20 MPa. The range of disturbance extended at least as far as the thermal 1°C disturbance.

The repository-scale analytical thermal and thermo-mechanical studies described above, regard temperatures and stresses averaged over large volumes without consideration of repository openings or the spatial distribution of heat sources. Numerical calculations performed at the tunnel scale (or at the deposition hole scale) give local temperature increases in the order of 50°C /Ageskog and Jansson 1999/ and stress increases in the order of 50 MPa close to the canister /Hakami and Olofsson 2000/.

The above analyses imply that the temperature change will affect the stress field in a fairly large region around the repository. However, since the thermal stress change is time continuous and relatively modest away from the tunnel, the stress change due to temperature changes was judged unnecessary to consider further in the present study.

5.5 Pore pressure changes

The normal effective stress across a fracture is dependent on the pore pressure in the rock mass. An increase in the pore pressure will result in a reduction of the shear strength. Time-dependent pore pressure changes in fractures around a deposition tunnel could, therefore, result in apparent creep movements.

The pore pressure around a repository will be a function of the groundwater table at the site and the drainage into the excavated opening. Some time (in the order of 10 years, /SKB 1999/) after repository closure, when the backfilled deposition tunnels have been re-saturated, the pore pressure will start to recover. Eventually the initial undisturbed conditions will be restored. The influence on the pore pressure from the elevated temperature can probably be neglected since the water can be assumed to be practically unconfined.

Considerable changes in the pore pressure may be expected if future glaciations occur. An ice cover of 2–3 km thickness may result in a pore pressure increase of 20–30 MPa in the rock mass. The load of an ice cover will also affect the in situ stress state, which will increase with approximately the same order of magnitude in the vertical direction. However, the glaciation case was judge unnecessary to consider in the present study for similar reasons as the thermal stress change, i.e. time continuous stress changes of a relatively modest magnitude.

5.6 Alteration of fracture properties

Alteration of fracture properties around a deposition tunnel could arise due to both chemical and biological weathering. Chemical weathering is expected mainly during the construction phase when water is percolating into the tunnel and air has unimpeded access to the tunnel. After the tunnel has been backfilled and sealed, future alteration in the fractures ought to be restricted to mainly biological weathering by anaerobic bacterial processes. Their ability to attack and change the mechanical properties of the joint surface and the infilling has not been investigated closer by the current study.

Weathering of joint infillings could cause a reduction in the rock mass shear strength with subsequent creep movements. Supposing, as suggested above, that it arises mainly during the construction phase, the zone of disturbance should coincide on the whole with the EDZ. This assumption is based on the fact that in this zone the frequency of open joints is high, which facilitates water movement and air penetration. However, since the period of time is relatively short the action of weathering is presumed to be limited and the fracture shear strength on the whole uninfluenced.

5.7 Conclusions

The review of disturbance factors that may trigger creep movements if the threshold stress/strength ratio is exceeded, shows that it is the extension of the EDZ and the stress redistribution zone that have to be considered, while the zone of temperature changes and pore pressure changes may be ignored. An outer perimeter may be determined by the stress redistribution. Concerning these zones the following conclusions can be drawn:

- Within the EDZ and joint alteration zone, a distance of approximately 0.5 radii, it is reasonable to expect a larger creep rate due to reduced rock mass shear strength. A way to consider this is to reduce the threshold ratio close to zero, within the actual region. However, the contribution of the tunnel convergence due to this reduction is limited, since the rock volume involved in this zone is small.
- The zone determined by the stress redistribution also has a relatively distinct and limited extent. With a conservative approach it is reasonable to assume that creep movements will arise within a distance of 6 radii from the opening, taking into account the stress redistribution.
- The zone of the temperature changes and the pore pressure changes are ignored, since these stress changes are time continuous changes of a relatively modest magnitude.

6 Conclusions

The following conclusions have been drawn concerning rock mass creep in general:

- The assumption that rock mass creep is almost entirely due to movements along rock fractures seems to be well supported in the literature surveyed here.
- Creep tests on clean and filled discontinuities indicate that it is mainly in filled discontinuities, and, in particular clay filled joints that creep movements may arise.
- The creep rate is not only a function of the shear stress to shear strength ratio, but also of the absolute values of the shear and normal stresses. This implies that it is mainly laboratory tests performed at a similar stress magnitude to that found in the real rock mass that are relevant for the forecast of creep movements.

Regarding threshold shear stress/strength ratios the following conclusion is drawn:

- Reported results confirm the occurrence of threshold values, related to the applied shear stress/ strength ratio, such that no creep takes place unless the threshold ratio is exceeded. A stress/ strength ratio of 30% for unfilled discontinuities and a ratio of 10% for filled discontinuities seem to be reasonable and conservative estimates.

It is possible that creep regions of different extents may arise around a deposition tunnel. Respecting these regions the following are concluded:

- Within the EDZ and joint alteration zone, a distance of approximately 0.5 radii, it is reasonable to presume that a larger creep rate may occur due to reduced rock mass shear strength. A way to consider this is to reduce the threshold ratio close to zero, within the actual region. However, the contribution of the tunnel convergence due to this reduction is limited, since the rock volume involved is small.
- Taking into account the stress redistribution, i.e. mechanically triggered movements, creep movements may arise within a distance of approximately 6 radii from the opening.
- The regions of temperature and pore pressure changes are consider unnecessary to consider, since these stress changes are time continuous changes of a relatively modest magnitude.

7 Recommendations

As indicated in the introduction to the report, it is possible to perform bounding estimates of creep movements with the use of numerical codes with standard built-in elasto-plastic models. By reducing the shear strength of the fractures a relaxation of fracture shear loads is reached. The accumulated displacements may then represent the maximum possible effects of creep that can take place in a jointed rock mass without consideration of the actual time it takes to reach those displacements. Results by /Glamheden et al. 2001/ from a pilot study performed using this technique are presented in the Figure 7-1. The backfill of the tunnel in the study was only considered as a friction material. The friction angle was reduced in 6 steps down to a value of 2.5 degrees as an extreme value for the case of joints with a thick clay filling. This type of infilling is rare in Swedish hard rocks at repository depth (500 m) and according to the site description of Forsmark, only few occurrences of clay filling were observed along fractures in zones /SKB 2008/.

As an illustration of how to interpret the results, the threshold values presented in the above conclusions are applied and illustrated in the diagram below. The initial shear strength, determined by the joint friction angle (30 degrees), is reduced to 30% of the initial strength for unfilled discontinuities (9 degrees) and to 10% for filled discontinuities (3 degrees).

The result is a tunnel displacement of approximately 15 mm of creep along unfilled fractures (the intersection between the spotted line and the curve of the wall convergence) and approximately 55 mm for filled fractures (the intersection between the dashed line and the curve of the wall convergence).

Based on the facts revealed by the literature survey, it is recommended that bounding estimates of the convergence are investigated for a deposition tunnel and associated deposition holes. The numerical modelling should include disturbance that may trigger creep movements in a rock volume limited by the stress redistribution zone around a deposition tunnel. In addition to the reduction of the friction angle to simulate creep movements, the modelling should also include a sensitivity analysis regarding joint geometry and joint dilation angle, which indirectly influence the rock mass shear strength.

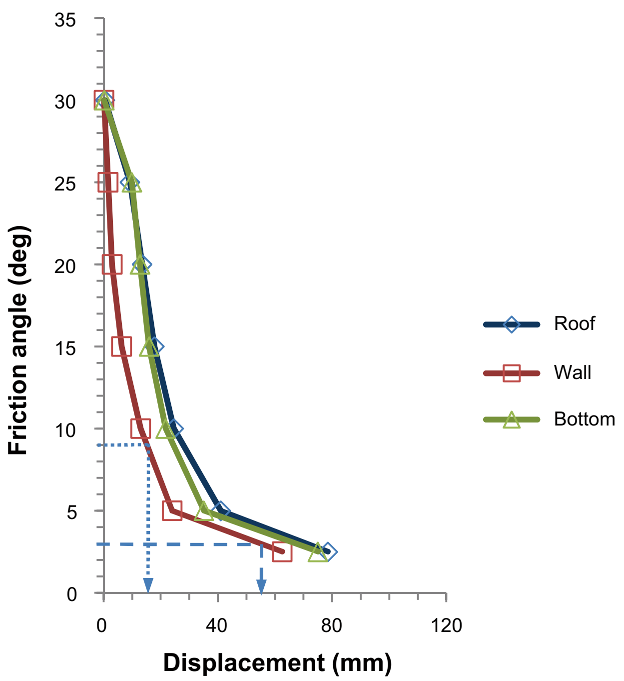

Figure 7-1. *Displacement versus friction angle from time-record points on the tunnel periphery in a numerical model. The results represent a tunnel where the backfill is considered only as a friction material. The arrows mark the estimated creep movements from the threshold values based on the literature survey. (Modified after /Glamheden et al. 2001/).*

8 References

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Symbols and abbreviations

Roman letters

a, b	empirical creep coefficients
A, B, C	empirical creep constant
c	joint cohesive strength
D_{ss}	steady state creep rate
i	peak dilatation angle
k_1, k_2	joint shear stiffness
K	spring stiffness
K_1, K_2	first and second spring stiffness element
M	empirical creep constant
n	logarithmic creep constant
R	shear stress ratio
t	time
T	clay content
v	creep velocity
v_1, v_2, v_3	creep velocity in primary, secondary and tertiary phase
Δu	shear displacement

Greek letters

ε_c	compressive strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	primary, secondary and tertiary creep
ϕ_b	basic friction angle
ϕ_p	peak friction angle
ϕ_r	residual friction angle
η	dashpot viscosity
η_1, η_2	first and second dashpot viscosity
μ	coefficient of static friction
μ_0	coefficient of initial friction
θ	joint orientation in relation to the applied stress
σ_c	uniaxial compressive strength of intact rock
σ_n	effective normal stress
$\sigma_1, \sigma_2, \sigma_3$	major, intermediate and minor principal stress
τ	shear stress
τ_p	joint peak shear strength
τ_r	joint residual shear strength
τ_s	joint shear strength

Abbreviations

3DEC	3 Dimensional Distinct Element Code
EDZ	Excavation Damage Zone
HRL	Hard Rock Laboratory
UDEC	Universal Distinct Element Code
URL	Underground Research Laboratory
ZEDEX	Zone of Excavation Disturbance Experiment