The strength of massive Lac du Bonnet granite around underground openings*

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Abstract

Tunnels excavated in massive unfractured Lac du Bonnet granite at the 420 Level of the Underground Research Laboratory showed typical signs of instability, i.e., spalling, slabbing, notch development. Two-dimensional elastic stress analyses of the failed tunnels indicated that failure was occurring at stress levels of about 100 MPa. A laboratory testing program was carried out using conventional unconfined-compression tests and triaxial-compression tests. In addition, uniaxial and biaxial tests of physical models containing circular holes were also conducted. All the laboratory tests indicated that the laboratory strength of Lac du Bonnet granite was twice the calculated stress level at which failure occurred. This result led to a study of massive rock strength using laboratory data and field observations.

In classical geotechnics, the shear strength of a rock is regarded as made up of two components, intrinsic strength or cohesion, and frictional strength. Laboratory tests were carried out which showed that initially, when rock deformations are essentially elastic, there is a maximum cohesive strength which is about 0.7-0.8 of the standard laboratory unconfined compressive strength. As the loads increase above this maximum cohesive strength, friction is increasingly mobilized and the associated nonelastic displacements damage the cohesion. Consequently at displacements near the peak strength, i.e., when friction is fully mobilized, approximately 70% of the initial cohesion has been lost. The laboratory tests showed that most of the cohesion loss results from very small displacements. The loss in cohesion was modelled using the Griffith locus based on a sliding crack model.

Microseismic monitoring of a circular test tunnel excavated on the 420 Level of the Underground Research Laboratory revealed that considerable damage, i.e., cracking, was occurring near the face of the advancing tunnel. Three dimensional numerical stress analyses were carried out to investigate the loading path near the face of the test tunnel. The analyses showed that the loading path exceeded the crack initiation stress measured in the laboratory tests but that the loading path did not exceed the initial cohesion values measured in the laboratory tests. Thus the loading path stress magnitudes were not sufficient to mobilize friction.

The effect of stress rotation, near the advancing tunnel face, was also investigated since it has been demonstrated, for tensile loading, that cracks can be made to grow at a constant load by rotating the direction of the applied load. Three-dimensional numerical stress analyses showed that the principal stress directions near the face of the tunnel rotate as the tunnel advances. It is proposed that the rotation of the stresses near the face amplifies the damage *in situ* and that this damage is equivalent to the damage in the laboratory tests in which the cohesion was reduced after only small displacements by 70%. Thus, when the maximum principal stress magnitude is above the crack initiation stress, the maximum cohesion *in situ*, that can be relied on, is only 50% or more of that measured in the laboratory.

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Two-dimensional modelling of the failure process was carried out using a phenomenological approach and a discrete fracture approach. The phenomenological approach uses a degraded strength to simulate the damage that has occurred near the advancing tunnel face. This approach, although practical, has several limitations, the most significant of which is the assumption that the entire rock mass strength around the tunnel has been degraded. The discrete fracture approach was carried out using a preliminary version of the finite element code InSight^{2D} designed to model fracture growth in compression. A major advantage of the discrete fracture method is that it does not require the rock mass strength to be degraded. This approach holds much promise and captures one of the key physical processes, i.e. slabbing, observed around failing underground openings in brittle rocks.

The major contributions to our understanding of strength and failure that have resulted from the investigations and analyses carried out during the course of this thesis are noted below:

- Samples obtained from a pre-stressed medium were damaged by the sampling process when the far field stress exceeded about $0.1\sigma_c$.
- The effects of scale, loading-rate and moisture on intact rock strength is minimal and do not explain the observed strength reduction investigated in this thesis.
- Maximum friction and maximum cohesion are not mobilized at the same displacements. By the time friction is fully mobilized a significant portion of the maximum cohesion in the sample has been lost. The Griffith locus was successfully used to model the mobilization of friction and the loss of cohesion.
- Cracking around underground openings initiates at about the same stress level as crack initiation in the laboratory tests. The zone of crack initiation, with $\phi = 0$, appears to define the limit of progressive failure around the openings at the 420 Level of the Underground Research Laboratory.
- The loading path of rock near the tunnel face suggests that stress rotation in the areas of maximum tangential stress concentration is a significant contributor to the localized degradation of the rock mass cohesive strength. Failure initiates in these locally damaged areas when the maximum tangential stress reaches about 100 MPa.
- Failure around the underground openings at the 420 Level of the Underground Research Laboratory initiates at a point, called the process zone, and is progressive. Failure occurs as a slabbing mechanism and stops when the geometry of the notch becomes convex and the process zone becomes contained.
- Modelling of the extent of failure cannot be captured by plane-strain modelling unless some effort is made to account for the initiation of failure at a point near the tunnel face before plane strain conditions are reached.

1 Introduction

1.1 Problem Statement and Research Activities

The nuclear industry is faced with assessing the concept of deep geological disposal of nuclear waste. One of the preferred characteristics for a suitable disposal site is low permeability. Hence, unlike the mining industry, which usually creates excavations in highly fractured rock masses with major faults and geologic boundaries, the nuclear industry is looking for uniform, stable conditions in rock masses with minimum faulting and fracturing. Thus, the shafts and tunnels for a waste repository may be excavated in nearly intact rock. AECL Research, which is charged with the responsibility for assessing the geologic disposal concept in plutonic rocks in Canada, has constructed the Underground Research Laboratory (URL) near Lac du Bonnet in south-eastern Manitoba. Portions of this facility are constructed in massive unfractured granite.

The tunnels excavated at the URL at a depth of 420 m in massive unfractured granite, displayed extensive spalling and slabbing during construction. Preliminary stress analyses of these tunnels indicated that the spalling and slabbing occurred where the maximum tangential stress on the boundary of the tunnels exceeded approximately 80–100 MPa. Thus the *in situ* strength around the perimeter of the tunnels is about 100 MPa or approximately 1/2 the unconfined compressive strength of intact rock samples tested in the laboratory. Similar findings have been reported elsewhere [1, 2, 3, 4]. However, in those cases explanations related to scale effects, heterogeneous materials and incipient fractures, were offered as reasons for the observed strength reduction. At the URL, however, the rock mass is relatively homogeneous and isotropic, elastic and fracture free. Thus the usual reasons put forth for the observed strength reduction, including scale effects, can not be supported at the URL. The inability of the classic stress-strength approach to predict tunnel performance at the 420 Level of the URL prompted the research described in this thesis.

1.1.1 Finding A Solution

When comparing measured laboratory strength to calculated stresses around tunnels, two important steps have to be carried out: 1) establish the far-field *in situ* stresses required to calculate the stresses around the tunnel, and 2) establish how the laboratory strength can be translated to a field strength. One additional step which is considered in this thesis is to establish if the loading path around the tunnel is similar to the loading path in the laboratory tests which were used to establish the in situ strength. In addition to these steps, one must also consider the assumptions employed in the numerical analyses. The rock mass at the URL is relatively homogeneous, isotropic, and fracture free, consequently, the analyses which appear most appropriate are ones which represent a continuous, homogeneous, isotropic, linear, elastic (CHILE) material. Thus, as a starting point, the calculated stresses around the URL tunnels are based on linear elastic analysis.

The first step in determining the stresses at the boundary of the tunnels at the URL was to establish the far field *in situ* stresses. It is well known that the measurement of *in situ* stress is fraught with difficulties. Accordingly, it becomes essential to establish the *in situ* stress magnitudes and direction with some reasonable degree of confidence. Although overcoring and hydraulic fracturing are traditional methods for determining the *in situ* stress state at a point, the convergence method provides the stress state at the tunnel scale. Martin et al [5] demonstrated that this type of stress measurement was considerably more reliable than the information obtained from overcore measurements at the URL.

The second step in the process was to establish the laboratory strength for the rock mass in which the tunnels will be excavated. In massive granite, this first appears as a routine task. Testing of Lac du Bonnet granite has been on-going since about 1980 by the University of Manitoba and AECL. Samples have been collected from two sources, Cold Spring Quarry and the URL. Preliminary compilation of the tests results from these two sources indicated that the strength of the granite varied spatially and also decreased considerably with depth at the URL. This latter finding was rather surprising because *in situ*, the quality of the rock mass improves with depth, i.e., the rock mass changes from a jointed rock mass near surface to a massive rock mass at depth. Thus in order to establish the likely *in situ* strength at the 420 Level of the URL, the possibility of sample disturbance needed to be explored.

Another important issue is translating the laboratory strength to the field strength. In the initial stress-strength analysis the laboratory unconfined compressive strength was compared to the calculated maximum tangential stress on the perimeter of the tunnel. However, it is well known that the short-term unconfined compressive strength is somewhat higher than the long-term strength of intact rock. The long-term strength of Lac du Bonnet granite has been determined by Schmidtke and Lajtai [6] to be about 130 MPa. However, this still does not explain failure occurring at stresses around 100 MPa. Interestingly, the 130 MPa long-term strength is projected from tests conducted over one month's duration and represents the predicted strength for times greater than 1000 years. The long-term strength from the one month tests was 155 MPa. Because failure was occurring as soon as the excavations were created it does not appear that the long-term strength plays a significant role in the observed strength reduction around the tunnels. In addition to the long-term strength, there is also the issue of scale (volume) effects, loading-rate effects, moisture effects and loading path effects. All of these effects are explored in this thesis.

As part of the study of the strength of Lac du Bonnet granite, physical model studies were also carried out. The initial stress-strength analysis compared the laboratory uniaxial compressive strength to the tangential stress around a tunnel. However, the stress at the boundary of a tunnel is not truly uniaxial because of the tunnel curvature and the load parallel to the axis of the tunnel. Thus in order to assess the strength around a tunnel granite blocks containing a circular opening were loaded in uniaxial or biaxial compression and cracking carefully monitored.

The next step in this research was to consider the role the loading path around the tunnel might play in determining the strength of the rock. Talebi and Young [7] established, by microseismic monitoring, that cracking, i.e., damage, was occurring ahead of the shaft excavated from the 240 Level to the 420 Level at the URL, although failure around the shaft was not observed in the areas where cracking had occurred. Observations of the failure process on the 420 Level indicated that it was progressive, i.e., initiating at a point and progressing into a typical well-bore breakout geometry. Thus, in order to understand the role of progressive failure a series of damage-controlled laboratory tests were conducted which enabled the cohesion and frictional strength components to be isolated and tracked as crack damage accumulated in the test specimen. This unique testing method indicated that as crack damage accumulates, at stress levels below the peak strength, cohesion is lost at a very rapid rate. Thus in a damaged sample, the cohesion can be as low as about half the cohesion measured in an undamaged sample. The strength at the boundary of an underground opening is essentially due to rock cohesion, thus these laboratory tests demonstrated that if damage is occurring around an advancing tunnel face the rock strength will be reduced to less than half the unconfined compressive strength.

The final stage of this research was to examine in detail how the loading path around an advancing tunnel face, causes damage and to attempt to model the process. Two and three dimensional numerical analyses were carried out. Finally numerical analyses were used to simulate the failure process around a tunnel at the 420 level of the URL. Two approaches to modelling the failure process were used, a phenomenological approach and a discrete fracture approach. Both methods, demonstrate that progressive failure around the tunnel can be modelled reasonably well once the mechanism of failure is thoroughly understood.

In summary, the primary objective of this thesis was to determine why the strength of massive granite from back analysis of tunnels excavated at the 420 Level of the Underground Research Laboratory was about 100 MPa when the the reported laboratory unconfined compressive strength of the granite was around 200 MPa. This objective was met by investigating the effects of scale, loading rate, moisture, damage and loading path on the rock strength.

1.2 Organization Of Thesis

This thesis examines the strength of openings in massive Lac du Bonnet granite at the Underground Research Laboratory at depths between 240 and 420 m. Chapter 2 provides the geotechnical setting for the URL. The *in situ* stress state at the 420 Level of the URL is established and the effects of this stress state on the properties of samples tested in the laboratory is described.

Extensive testing of Lac du Bonnet granite has been carried out over the past ten years. Chapter 3 provides a summary of those test results. It is commonly implied that scale (volume) effects is the major reason for the reduction in strength from the laboratory measured value to the *in situ* strength. Scale effects, loading rate effects, moisture, and long-term strength are all investigated and reviewed in arriving at an estimate of the likely *in situ* strength.

Interpretation of laboratory test results requires an understanding of the failure process. Chapter 4 provides a review of laboratory testing in compression and establishes the key fracture parameters in compression testing. Chapter 4 then examines the progressive failure of brittle rock. The classical strength envelope for a brittle rock assumes that full cohesion and full friction are mobilized simultaneously. Results from damage-controlled laboratory tests are used to investigate this assumption. A model, based on energy concepts, is developed for progressive failure and compared to the laboratory results.

In Chapter 5 it is demonstrated that the back calculated strength around the excavations at the 420 Level of the Underground Research Laboratory is about half the laboratory unconfined compressive strength. Microseismic monitoring of a test tunnel shows that damage is occurring to the rock immediately around the face of the advancing tunnel. Mechanisms which could cause this damage are investigated, using numerical simulations, and compared to the laboratory test results.

Chapter 6 describes two approaches, the phenomenological approach and the discrete fracture approach, to modelling progressive failure around the openings at the URL. The phenomenological approach requires no understanding of the failure process whereas the discrete fracture method requires a thorough understanding of the failure process.

A discussion and summary of the thesis is provided in Chapter 7.

2 Summary and Conclusions

The major objective of this thesis was to investigate why the strength of massive granite from back analysis of tunnels excavated at the 420 Level of the Underground Research Laboratory was about 100 MPa when the reported laboratory unconfined compressive strength, σ_c, of the granite was around 200 MPa. In the past, strength reductions found around underground openings have generally been attributed to scale effects, i.e., the strength decreases as the volume of the sample increases. At the URL it was obvious that scale effects could not explain the strength reduction because a 100-mm diameter borehole displayed the same failure as a 3.5-m diameter tunnel.

The testing programs carried out as part of this thesis revealed that excavating tunnels at the 420 Level causes damage around the advancing tunnel face and that this damage reduces the rock strength. The following is a summary of the progressive damage/failure process found around the underground openings at the URL.

2.1 Laboratory Progressive Failure

Analyses of unconfined compression test results revealed that cracking in compression begins at about $0.4\sigma_c$. This cracking continues as the load increases until the crack damage stress is reached at about $0.8\sigma_c$, at which point the sample starts to dilate. The load can be temporarily increased above this stress level but rock cannot sustain this load for any significant length of time. Thus, the crack damage stress is the true strength or cohesion of the rock in unconfined compression. This concept is in complete agreement with the concrete industry which defines the long-term strength of concrete at the crack damage stress.

The laboratory properties of Lac du Bonnet granite were reviewed in Chapter 3. In Chapter 4 it was shown that the crack damage stress of Lac du Bonnet granite was not significantly influenced by scale effects and that properly conducted laboratory tests would fail at the crack damage stress, again supporting the notion that the traditional peak strength is an artifact of the test loading conditions. Also, it was shown that although the initial cracks during a compression test form parallel to the direction of the maximum applied load, the final failure surface is one which is inclined at about 23° to the direction of the applied load. This inclined failure surface was more pronounced in the tests which failed near or at the crack damage stress, suggesting that the axial splitting mode of failure may also be an artifact of the test conditions.

Damage-controlled laboratory tests were carried out to investigate the influence of crack volume on the crack damage stress. The testing program demonstrated that the crack damage stress decreased significantly to a crack damage threshold as damage accumulated in the sample. This threshold value coincided with the crack initiation stress for unconfined tests but as the confining stress increased the crack damage threshold value was always above the crack initiation stress.

The locus of crack damage stress with increasing damage was modelled using the Griffith locus of sliding and found to be in excellent agreement. The Griffith locus, which is based on fracture mechanics, was first introduced by Berry [8, 9] for tension and Cook [10] for compression. In developing the Griffith locus for triaxial compression, it became obvious that the strength of the material described by the Griffith locus had two strength components — cohesion and friction, but only the cohesion was controlled by the fracture mechanics properties such as fracture surface energy and crack length.

It is generally assumed that cohesion and friction are mobilized at the same displacements. The damage-controlled tests and the Griffith locus provided a means of separating the portion of the strength due to cohesion, basic friction and friction caused by roughness. At the instant the crack damage stress is first reached in a compression test the strength is essentially cohesion. At loads beyond the crack damage stress, friction becomes mobilized. Unfortunately, in the process of mobilizing friction, significant cohesion is lost. When displacements are large this cohesion loss can amount to 50% or more of the initial cohesion.

Damage-controlled tests of Indiana limestone and Rocanville potash showed similar results to those from the granite testing program, the major difference being the slope of the Griffith locus in the softer rocks. It appears that the loss in cohesion in potash would not be nearly as rapid as in granite.

The concept of cohesion loss as a rock is damaged suggests that the traditional failure envelopes for rock around underground openings are not adequate. At the boundary of the openings the strength could be reduced to less than 50% or more of the initial cohesion. However, at some distance from the boundary, where $\sigma_3 > 0$ the strength of the undamaged rock is not degraded. Thus around the opening the failure envelope is very steep, being equal to $\approx 0.3\sigma_c$ at $\sigma_3 = 0$ and rising steeply to the initial crack damage stress when σ_3 is slightly greater than zero.

2.2 In Situ Progressive Failure

The concept of reducing cohesion with increasing damage was explored through the phenomenon of sample disturbance. The process of extracting a sample from a medium stressed above a critical level causes stress-induced microcracking which in the extreme case produces core discing. It was shown that the strength, i.e., cohesion, of samples with extensive microcrack damage, is significantly reduced when compared to undamaged samples. What is most important is that it is only the cohesion component of the strength envelope which is affected by the damage and that this cohesion cannot be regained by applying a confining stress. The laboratory testing program revealed that the properties of Lac du Bonnet granite varied, depending on the location of the samples and that the strength of the samples from Cold Spring Quarry was noticeably higher than the strength of the near surface samples from the URL. This variation in rock properties was attributed to the crack density of the laboratory samples and not to the crack density in situ.

Physical models of openings of various diameters were loaded in uniaxial and biaxial compression. These model studies were carried out to examine the strength around circular openings. In the physical model studies the tangential stress required for spalling failure to occur around circular openings was about 220 MPa. This is considerably higher then the observed *in situ* strength. Thus physical model studies could not replicate the reduced strength observed *in situ*. Hence it was concluded that the reduced strength must be caused by something which is unique to the tunnel excavation *in situ*.

Microseismic monitoring of a 3.5-m diameter test tunnel revealed that considerable cracking was occurring around the face of the advancing tunnel. This cracking was found to occur to a depth of about $1.3a$ relative to the centre of the tunnel, where a is the tunnel radius. The microseismic events ahead of the tunnel face represent the initiation of cracking and not the failure strength.

Stress analysis revealed that the stress magnitudes at the location of these microseismic events agreed very well with the crack initiation stress obtained from the laboratory testing program. The crack initiation stresses for the microseismic events also fitted the Cook envelope with $\phi = 0$, again suggesting friction had not been mobilized. Although the Griffith failure envelope for compression is suppose to represent the initiation of fracture, it gave a poor fit to the *in situ* microseismic cracking stress.

The *in situ* crack initiation envelope developed from the microseismic events was then used to evaluate the stress path at various locations around the tunnel as the tunnel face advances. It was shown that the stresses around the tunnel did not exceed the crack initiation envelope beyond about $1.5a$ from the center of the tunnel, where a is the tunnel radius. Thus at distances less then $1.5a$ cracking should be occurring. This agreed quite closely with the locations of the microseismic events and the measured depth of spalling around the Mine-by test tunnel.

The unconfined *in situ* compressive strength of Lac du Bonnet granite is estimated at 150 MPa. This is considerably less than the short-term unconfined compressive strength, σ_c, of 220 MPa for undamaged samples from Cold Spring Quarry and has been degraded to reflect the laboratory findings discussed in Chapter 4. The 3-dimensional numerical analyses showed that the tunnel stresses near the face of an advancing tunnel did not exceed 150 MPa and at the point on the tunnel boundary where failure was observed the calculated maximum tangential stress was only about 80-100 MPa. Thus from the numerical analyses and the best estimate of the *in situ* strength it appeared that the loading path followed in the laboratory tests was not the loading path followed *in situ*, because according to the laboratory results failure should not have occurred.

In addition to the stress path around the tunnel, the effects of stress rotation were also considered. Wu and Pollard [11] demonstrated, for tension, that a stress rotation could influence the direction and origin of crack growth and cause additional crack growth without increasing the load. It was shown that the compressive stress near the tunnel face also rotates and it is proposed that this stress rotation contributes to the loss in cohesion. In the Mine-by test tunnel the σ_1 stress magnitude increases above the crack initiation stress of 70 MPa at 0.25 m ahead of the tunnel face, to about 100 MPa at 0.5 m inside the tunnel. Because the stress directions were rotated by the advancing tunnel, it is postulated that the crack length is increased above what would be expected with no rotation. The result of this additional crack growth is reduced cohesion. Interestingly, it is only at the maximum tangential stress concentration that the stress rotates to its maximum value ($\approx 25°$) at the tunnel face and then rotates back to its original position as the tunnel advances. At other positions around the tunnel perimeter the stress either does not rotate or does not rotate back to its original position. It is suggested that this complex loading path is responsible for the damage, i.e., loss in cohesion, in the region of the maximum tangential stress concentration.

Observations made during the excavation of the Mine-by test tunnel revealed the failure process initiated at the maximum tangential stress concentration and that in this zone the rock failed by crushing. After this crushing occurred thin slabs were released violently from around the crushed zone. This crushed zone can be considered equivalent to the "process zone" at the tip of an advancing crack. Once the slabbing process initiates, it continues as the tunnel face is advanced. When the notch finally stabilizes the geometry of the notch displays a characteristic convex shape with the process zone at the tip of the notch.

2.3 Modelling the Failure Process

Physical model studies of a circular hole in blocks of Lac du Bonnet granite were carried out with and without confining stress. The unconfined samples revealed that three types of cracks occurred around the circular openings: primary crack, remote crack and the spalling crack. The primary crack occurred first followed by the remote crack which was subsequently followed by the spalling crack. The sequencing of the remote and spalling cracks was reversed for the confined case. This change in fracture sequence for the confined case occurs because the primary crack does not grow very far compared to the unconfined case. Thus the sequence of fracturing is influenced by the stress redistribution which occurs as a result of the initial primary crack. In both the unconfined and confined physical models the spalling cracks formed when the maximum tangential stress on the boundary of the circular hole was slightly greater then the unconfined compressive strength.

Two approaches were used in the modelling of the failure process around underground openings: the phenomenological approach and the discrete fracture approach. The phenomenological approach made no assumption of the mechanisms involved in the failure process. It simply assumes a degraded rock strength everywhere around the opening. In this case the strength was assumed to be approximately equal to the initial crack damage stress envelope obtained from the testing of the damaged samples from the 420 Level of the URL. It was reasoned that the damage process around the tunnel is similar to the damage process in extracting a sample, and therefore, the initial crack damage stress of the damaged samples would be a reasonable estimate of damaged *in situ* strength. A strength to stress ratio is used to locate the "failed zones". In this modelling approach an elastic-brittle material behaviour is assumed such that once the failed zones are located, then the tunnel shape is modified to reflect the removal of this failed material, i.e., the slabbing process. After the geometry is modified the elastic analysis is rerun. The process is repeated until a reasonably stable geometry is achieved. For this modelling approach to work, some idea of the final stable shape is required. Also, the approach is sensitive to the amount of the material removed in each step. In this simplified approach, a notch which extends about $2a$ from the tunnel centre is produced. This is considerably deeper than the notches observed at the URL.

After observing the crack growth in the physical models, it was postulated that the process zone must be a key factor in modelling the notch development. The 2-dimensional finite element program InSight^{2D}, containing an empirical compressive crack model, was used to investigate the spalling process. The initial far-field stresses were set to provide a maximum tangential stress of 100 MPa, the stress level at which failure was first observed. The process zone, which was observed *in situ*, was represented in the InSight^{2D} model by assigning a small region in the zone of maximum tangential stress a reduced modulus equivalent to about 1/12 of the undamaged modulus of 60 GPa. This softened process zone caused tension to appear near the boundary of the tunnel on both sides of the process zone. A compressive crack was then manually inserted into this region of tension, parallel to the direction of the maximum principal stress. The far-field stress was then increased to provide the maximum tangential stress given by the plane strain conditions. The compressive crack, with the new stress conditions, became unstable. The crack was then extended until it reached the process zone, at which point the other crack tip still remained unstable. Crack propagation continued until the crack tip approached the boundary of the tunnel. Although the crack propagation in InSight^{2D} is entirely manual, the program appears to capture all the key elements observed during the failure process *in situ*: a process zone, slabbing, and convex curvature near the notch tip. The major advantage of the discrete fracture approach is that the rock mass strength is not degraded as with the phenomenological approach.

3 Conclusions

Listed below is a brief summary of the key contributions and conclusions of this thesis towards an advancement in the understanding of rock failure and rock strength around underground openings.

1. The broad distribution of stresses in a large rock mass can be explained using elastic theory.

2. Samples obtained from a pre-stressed medium may be damaged by the sampling process. This damage, which starts to occur when the far field stress exceeds about $0.15\sigma_c$, affects the strength of the sample. In particular, the damage reduces the cohesion but does not affect the friction. Thus intact samples obtained from a pre-stressed rock mass may not necessarily represent the *in situ* intact strength.

3. The effects of scale, loading-rate and moisture on intact strength of granite is minimal and do not explain the observed strength reduction investigated in this thesis.

4. The laboratory unconfined compressive strength (σ_c) of granite is not a material property but is dependent on the loading conditions. The crack initiation and crack damage stress were found to be essentially independent of loading conditions and are considered material properties.

5. Cracking initiates in laboratory samples at about $0.4\sigma_c$ as stable crack growth parallel to the direction of the maximum applied load.

6. Sliding in a sample occurs at about 0.7-$0.8\sigma_c$ when the crack damage stress exceeds the cohesive strength. Initially, as sliding first starts the mechanism is unstable.

7. Damage-controlled tests can be used to map the locus of stress associated with crack initiation and sliding. The Griffith locus of sliding was applied to the sliding locus obtained from the damage-controlled tests and gave excellent agreement.

8. The results from the damage-controlled tests and the application of the Griffith locus demonstrated that friction and cohesion are not mobilized equally as a sample is strained. By the time the friction is fully mobilized a significant portion of the initial cohesion in the sample is lost. This implies that the prediction of failure around tunnels which are experiencing damage cannot be based on strength envelopes derived from traditional laboratory tests.

9. The Cook failure criterion was extended and the extension separated the Cook failure envelope in σ_1–σ_3 space into cohesion and friction with the cohesion as a function of crack length and fracture surface energy.

10. Cracking around underground openings initiates at about the same stress level as crack initiation in the laboratory tests. Like the laboratory test results, the crack initiation *in situ* suggests that the cracking is only affecting the cohesion of the rock mass. The zone of the crack initiation, with $\phi = 0$, appears to define the limit of progressive failure around the openings at the 420 Level of the Underground Research Laboratory.

11. The loading path of rock near the tunnel face suggests that stress rotation in the areas of maximum tangential stress concentration is a significant contributor to the localized degradation of the rock mass cohesive strength. Failure initiates in these locally damaged areas when the maximum tangential stress reaches about 100 MPa which is approximately 50% of σ_c.

12. Failure around the underground openings at the 420 Level of the Underground Research Laboratory initiates at a point, called the process zone, and is progressive. Failure occurs as a slabbing mechanism and stops when the geometry of the notch becomes convex and the process zone becomes contained. The depth of this type of failure at the Underground Research Laboratory is about $1.3a$, where a is the tunnel radius.

13. Modelling of the extent of failure cannot be captured by plane-strain modelling unless some effort is made to account for the initiation of failure at a point near the tunnel face before plane strain conditions are reached.

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