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Static and Cyclic Laboratory Testing of Brisbane Rocks

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Declaration by author

This thesis *is composed of my original work, and contains* no material previously published or written by another person, except where due reference has been made in the text. I have clearly stated the contribution by others to jointly-authored works that I have included in my thesis.

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Chapter 2

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Abstract

The use of underground facilities for purposes such as transportation tunnels, power station caverns, radioactive waste repositories, and water and gas storage is increasing. Fracture is a failure mechanism of brittle rocks that is of great importance to their performance for underground facilities. A major challenge in rock fracture mechanics research at its current stage of development is the extrapolation of parameter values measured at laboratory-scale on small, intact rock specimens to large-scale jointed rock masses. Understanding crack growth is therefore very important for assessing the stability of rock masses/materials and for the safe design of structures within rock masses. The aim of this thesis is to investigate the tensile strength parameters of rocks, such as the indirect tensile strength and fracture toughness value, under monotonic static and cyclic loading conditions. The indirect tensile strength of rocks was initially investigated using the standard Brazilian test. However, this was found to crush the brittle rocks tested, and steel loading arcs of different angles were assessed to obtain more reliable diametral tensile splitting cracks in disc specimens, as expected theoretically. The maximum recorded ultimate load and most appropriate tensile splitting crack was obtained with a loading arc angle $2\alpha = 30^\circ$. However, the highest standard deviation of the ultimate load was obtained from the standard Brazilian test results.

The most fundamental parameter in fracture mechanics is the fracture toughness, which describes the resistance of a material to crack propagation. Fracture toughness is an important material parameter that corresponds to the critical state of the stress intensity factor required for crack initiation and subsequent propagation. Both the static and cyclic loading tests were carried out on Cracked Chevron Notched Brazilian Disc (CCNBD) rock specimens for fracture toughness investigations. In order to yield consistent fracture toughness values, two test methods were

performed: CCNBD and short rod (SR) tests. After a series of experiments, the fracture toughness value obtained by the SR method was found to be higher than the value obtained by the CCNBD method. It was realised that the critical stress intensity factor (SIF) used in the formula for calculating the fracture toughness of the CCNBD specimens is a crucial factor in the method proposed by the ISRM (ISRM, 1995).

In the cyclic loading tests, the reduction in fracture toughness under increasing cyclic loading was found to be up to 48% of the static fracture toughness. The experimental results showed continuous irreversible damage accumulation at the tip of the chevron notched crack in the dynamic cyclic loading tests, strongly influenced by the loading amplitude. Contrary to the static tests, the cyclic tests produced much more crushed material in front of the tip of the chevron notched crack. On the other hand, the reduction in the indirect tensile strength (ITS) was found to be 30% of the static ITS with sinusoidal cyclic loading tests, whereas increasing cyclic loading with the same valley values cause a reduction of the ITS by up to 35 % of the static ITS.

In order to examine the major characteristics of the fatigue damage process at the microscopic level, scanning electron microscope (SEM) imaging was used. SEM investigations show that particle boundaries play an important role in rock fatigue. When compared with static rupture, the main differences are two-fold: (a) the number of fragments produced is much greater under cyclic loading than under static loading, and (b) intergranular cracks are formed due to grain loosening under cyclic loading. The main characteristic is particle breakage under cyclic loading, which probably starts at contacts between particles and is accompanied by the production of very small fragments, probably due to frictional sliding within the weak matrix. It is believed that point contacts at particle boundaries are regions of stress concentration (i.e. indenters). Transgranular cracks may emanate from these regions, and intergranular cracks sometimes pass through the contact points. This stage can be correlated with a steady progression of damage and produces a general “loosening” of the rock, which is a precursor to the formation of intergranular cracks.

Keywords

Brazilian indirect tensile test, CCNBD specimen, increasing cyclic loading, mode I rock fracturing, rock fracture toughness, rock fatigue, scanning electron microscopy

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Contents

Declaration by Author.....	ii
Abstract.....	v
List of Tables.....	x
List of Figures.....	xi
List of Abbreviations.....	xix
CHAPTER 1 INTRODUCTION.....	1
1.1. General.....	1
1.2 Scope of Research.....	2
1.3 Thesis Layout.....	3
CHAPTER 2 DEFORMATION AND FRACTURING OF BRITTLE ROCKS	5
2.1 Introduction.....	5
2.2 Theories of Brittle Fracture.....	7
2.2.1 Griffith's theory	8
2.2.2 Modified Griffith fracture stress criterion and linear elastic fracture mechanics	11
2.3 Initiation and Propagation of Cracks in Brittle Rocks	12
2.4 Tensile Strength of Rocks.....	16
2.5 Fracture Toughness of Rocks.....	20
2.5.1 Methods of mode I fracture toughness testing.....	21
2.5.2 Methods of mode II and mixed-mode I–II fracture toughness testing.....	24
2.6 Cyclic Loading and Rock Fatigue.....	28
2.6.1 Fatigue damage mechanism.....	35
2.7 Chapter Summary	40
CHAPTER 3 EQUIPMENT AND MATERIALS.....	41
3.1 Loading and Deformation Measurement Equipment.....	41
3.1.1 Cyclic loading tests	44
3.2 Sample Preparation	46
3.3 Tested Rocks.....	50
3.4 Scanning Electron Microscope	53
CHAPTER 4 DETERMINATION OF TENSILE STRENGTH OF ROCKS—METHODS AND RESULTS	56
4.1 Methods.....	56
4.1.1 Direct tension test	56

4.1.2 ISRM suggested indirect Brazilian test.....	59
4.1.3 Different angled loading arc tests	61
4.2 Results.....	62
4.2.1 Direct tensile strength	62
4.2.2 Indirect Brazilian tensile strength	64
4.2.3 Indirect tensile strength by loading arcs	67
4.2.4 Numerical modelling results	72
4.3 Discussion of Tensile Strength Results.....	78
4.3.1 Comparison between direct and indirect tensile strengths.....	79
4.3.2 Analytical and numerical studies	79
4.3.4 Comparison between experimental, analytical and numerical results	81
4.4 Chapter Summary	89
CHAPTER 5 DETERMINATION OF FRACTURE TOUGHNESS OF ROCKS—METHODS AND RESULTS.....	91
5.1 Methods.....	91
5.1.1 Cracked Chevron Notched Brazilian Disc test	91
5.1.2 Short Rod test.....	93
5.2 Results.....	96
5.2.1 K_{IC} determined by the CCNBD and SR methods for several Brisbane tuff types	97
5.2.2 K_{IIC} and mixed-mode fracturing of rocks determined by the CCNBD method.....	99
5.2.3 Numerical simulation results	107
5.3 Discussion of Fracture Toughness Results	118
5.3.1 Comparison of CCNBD and SR methods.....	118
5.3.2 Comparison between experimental and numerical results of mixed-mode fracturing ...	119
5.4 Chapter Summary	122
CHAPTER 6 DEGRADATION OF INDIRECT TENSILE STRENGTH OF ROCKS THROUGH CYCLIC LOADING.....	126
6.1 Sinusoidal Cyclic Tests.....	124
6.1.1 Determination of S—N curve for the fatigue limit of indirect Brazilian tensile strength of Brisbane Tuff	125
6.2 Effect of Increasing Cyclic Loading on Indirect Tensile Strength of Brisbane Tuff.....	127
6.3 Fatigue Damage Mechanism under Indirect Tensile Cyclic Loading	142
6.4 Chapter Summary	150
CHAPTER 7 DEGRADATION OF FRACTURE TOUGHNESS OF ROCKS THROUGH CYCLIC LOADING.....	152

7.1 Sinusoidal Cyclic Tests.....	152
7.1.1 Determination of S—N curve for fatigue limit of CCNBD fracture toughness of Brisbane Tuff	153
7.2 Effect of Increasing Cyclic Loading on Mode I Fracture Toughness.....	155
7.2.1 Effect of Cyclic Loading with Increasing Valley Values on Mode I Fracture Toughness	157
7.2.2 Effect of Cyclic Loading with Increasing Mean Values on Mode I Fracture Toughness	170
7.3 Behaviour of Inclined Chevron Notched Crack under Cyclic Loading.....	184
7.4 The Mechanism of Rock Fatigue Damage in Front of Chevron Notched Crack.....	206
7.5 Chapter Summary.....	224
CHAPTER 8 DISCUSSION.....	226
8.1 Indirect Tensile Rock Fracturing under Static Loading.....	226
8.2 Fracturing Modes and Influencing Factors	228
8.3 Indirect Tensile Strength Degradation under Cyclic Loading	230
8.4 Fracture Toughness Degradation under Cyclic Loading	231
CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS.....	232
9.1 Conclusions.....	232
9.2 Recommendations for Future Research	236
APPENDIX A Degradation of Indirect Tensile Strength of Rocks through Cyclic Loading.....	237
APPENDIX B Degradation of Fracture Toughness of Rocks through Cyclic Loading.....	243
APPENDIX C X-Ray Diffractometer (XRD) Analysis.....	246
APPENDIX D SEM Images.....	248
APPENDIX E Doyens & Inspirers in my Research.....	250

List of Tables

Table 3.1 Mechanical characterisation results for Brisbane tuff	52
Table 4.1 Results of direct tensile tests on Brisbane tuff.....	63
Table 4.2 Results of indirect tensile tests on Brisbane tuff disc specimens	65
Table 4.3 Results of indirect tensile tests using loading arcs	68
Table 4.4 Comparison of experimental, numerical and analytical results for Brisbane tuff	88
Table 5.1. Mode I fracture toughness values of Brisbane tuff obtained by SR and CCNBD tests ...	97
Table 5.2 Failure loads obtained for Brisbane tuff, granite and sandstone CCNBD samples	101
Table 5.3 β and θ values for Brisbane tuff, granite and sandstone samples	103
Table 6.1 Results of sinusoidal cyclic loading	126
Table 6.2 Results of Type I cyclic loading tests performed on Brazilian disc specimens	130
Table 6.3 Results of Type II cyclic loading tests performed on Brazilian disc specimens.....	133
Table 7.1 Results of sinusoidal cyclic loading.....	154
Table 7.2 Results of Type I cyclic loading tests on CCNBD specimens.....	158
Table 7.3 Results of Type II cyclic loading tests on CCNBD samples.....	172
Table 7.4 Failure load and number of cycles for tested CCNBD samples with inclined notch crack.....	186
Table 8.1 Comparison of experimental, numerical and analytical results for Brisbane tuff.....	227

List of Figures

Figure 2.1 Brittle fracture process in a rock crack is either stable or unstable, depending on the value of the crack critical energy release (G)	8
Figure 2.2 Three fracture modes and corresponding crack surface displacements	14
Figure 2.3 Crack geometry and inclination used in various crack modelling methods (a) inclined ellipse cracks (b) inclined cracks with zero width (c) axial ellipse cracks	14
Figure 2.4 Crack propagation paths under compression (a) and tension (b)	16
Figure 2.5 A disc specimen subjected to diametric distributed compression	19
Figure 2.6 Crack tip coordinates and stress state in terms of the Cartesian system	20
Figure 2.7 Chevron notched Short Rod specimen and associated notations	21
Figure 2.8 The CCNBD specimen geometry with recommended test setup	23
Figure 2.9 Valid geometrical ranges for fracture toughness testing (Fowell, 1994).	24
Figure 2.10 Sample geometry and principal loading setup for PTS-Test.....	25
Figure 2.11 State of load (a) and stress at the crack tip (b) for the shear fracture model (Rao <i>et al.</i> , 2003).....	26
Figure 2.12 CCBBD sample under diametral compression	27
Figure 2.13 Definitions of cyclic loading	30
Figure 2.14 Typical S—N graph for Indiana limestone (Costin & Holcom, 1981).....	30
Figure 2.15 Conceptual model of crack coalescence accompanied by large permanent axial strains (After Eberdhard <i>et al.</i> , 1999).....	37
Figure 2.16 Formation of lateral cracks at crack surface asperities.....	38
Figure 2.17 Intergranular and intragranular fractures in a cyclically broken marble specimen	39
Figure 2.18 Crystal cleavage fracture in a monotonically broken marble specimen.....	39
Figure 3.1 DAQ system	43
Figure 3.2 Three reading channels for load, axial/lateral strain and position.....	44
Figure 3.3 Sinusoidal cyclic loading—constant mean stress with constant amplitude	45
Figure 3.4 Increasing cyclic loading—increasing mean stress with constant amplitude	45
Figure 3.5 Grinding the ends of the UCS samples.....	47
Figure 3.6 Two successfully strain-gauged UCS samples.....	48
Figure 3.7 Valid geometrical ranges for CCNBD specimen	48
Figure 3.8 CCNBD specimen preparation equipment (a) diamond saw (b) electronic alignment equipment.....	49
Figure 3.9 Prepared CCNBD specimens	49
Figure 3.10 Photomicrograph of Feldspar (Fl) and Quartz (Q) mineral.....	51

Figure 3.11 Microcrystalline quartz cement (a) and Quartz (Q) mineral in microcrystalline quartz and primary cryptocrystalline cement (b)	51
Figure 3.12 Stress versus strain curve of a tested strain-gauged UCS sample	52
Figure 3.13 External components of the JEOL JMS-6460 LA SEM	54
Figure 4.1 (a) Prepared specimens before direct tensile testing, (b) direct tensile loading test apparatus	58
Figure 4.2 Loading jaws for indirect tensile Brazilian test	59
Figure 4.3 Brisbane tuff disc sample between standard Brazilian jaws	60
Figure 4.4 (a) Steel loading arcs (b) diagram of steel loading arcs and disc specimens	62
Figure 4.5 Tested samples (a) before caps are removed (b) after caps are removed.....	63
Figure 4.6 Stress–strain plots of direct tension tests.....	64
Figure 4.7 Stress–strain curves of standard Brazilian tests.....	66
Figure 4.8 Failed Brisbane tuff specimens (a) NST35 and (b) NST207 under standard Brazilian jaws	67
Figure 4.9 Failed Brisbane tuff samples under (a) Brazilian jaws (b) 15° loading arc (c) 20° loading arc (d) 30° loading arc.....	69
Figure 4.10 Failed sandstone samples (a) standard Brazilian jaws (b) 15° loading arc (c) 20° loading arc (d) 30° loading arc.....	70
Figure 4.11 Failed granite samples under (a) standard Brazilian jaws (b) 15° loading arc (c) 20° loading arc (d) 30° loading arc.....	71
Figure 4.12 Comparison of stress–strain plots for the standard Brazilian and the loading arc tests .	72
Figure 4.13 Standard Brazilian jaw simulations (a) Horizontal stress distribution (MPa) (b) Maximum principal stress distribution (+: tensile, -: compressive) c)Tensile (white) and compressive (orange) stress bars	74
Figure 4.14 15° loading arc simulations (a) Horizontal stress distribution (MPa) (+: tensile, -: compressive) (b) Tensile (white) and compressive (orange) stress bars	75
Figure 4.15 20° loading arc simulations (a) Horizontal stress distribution (MPa) (+: tensile, -: compressive) (b) Tensile (white) and compressive (orange) stress bars	76
Figure 4.16 30° loading arc simulations (a) Horizontal stress distribution (MPa) (+: tensile, -: compressive) (b) Tensile (white) and compressive (orange) stress bars	77
Figure 4.17 Tensile stress distributions along horizontal diameter of the disc.....	78
Figure 4.18 Analytical stress distribution along vertical diameter (BY) of the disc (+: tension, -: compression).....	80

Figure 4.19 Horizontal stress distribution along the vertical diameter (BY) of the disc (+: tension, -: compression).....	81
Figure 4.20 Three pre-existing cracks along the vertical diameter of disc.....	82
Figure 4.21 (a) Stress distribution in x direction, and (b) Tensile stress bars under 20° loading arc simulation for a disc with three pre-existing cracks (+: tension; -: compression).....	83
Figure 4.22 Simulated tensile stress distributions across the horizontal diameter of disc through the central pre-existing cracks	83
Figure 4.23 Progressive fracture process induced in (a) standard Brazilian jaws (b) 15° loading arcs (c) 20° loading arcs (d) 30° loading arcs.....	85
Figure 4.24 Fringe patterns of maximum principal stress induced for (a) standard Brazilian jaws (b) 15° loading arcs (c) 20° loading arcs (d) 30° loading arcs.....	86
Figure 4.25 Fringe patterns of minimum principal stress induced for (a) standard Brazilian jaws (b) 15° loading arcs (c) 20° loading arcs (d) 30° loading arcs	86
Figure 5.1 The CCNBD specimen geometry with recommended test fixture	92
Figure 5.2 Valid geometrical ranges of CCNBD specimens for fracture toughness testing	92
Figure 5.3 CCNBD specimen preparation equipment	93
Figure 5.4 (a), (b) Prepared CCNBD samples and (c) loading plates and CMOD transducer	94
Figure 5.5 Chevron notched Short Rod specimen and associated notations	95
Figure 5.6 Brisbane tuff Short Rod specimens before testing	96
Figure 5.7 (a) SR and (b) CCNBD specimens after testing.....	97
Figure 5.8 Dimensionless stress intensity factor for CSTBD and CCNBD specimens.....	99
Figure 5.9 Prepared CCNBD samples (a) Tuff (b) Granite and sandstone (c) Loading platens	100
Figure 5.10 CCBD sample under diametral compression	101
Figure 5.11 Tested CCNBD specimens (a) Brisbane tuff (b) granite and (c) sandstone.....	102
Figure 5.12 Crack displacement under mixed-mode loading of Brisbane tuff CCNBD samples ...	104
Figure 5.13 Crack displacement under mixed-mode loading of granite CCNBD samples	105
Figure 5.14 Crack displacement under mixed-mode loading of sandstone CCNBD samples	105
Figure 5.15 Maximum notch crack displacement under mixed-mode I-II loading for Brisbane tuff CCNBD samples.....	106
Figure 5.16 Maximum notch crack displacement under mixed-mode I-II loading for granite CCNBD samples.....	106
Figure 5.17 Maximum notch crack displacement under mixed-mode I-II loading for sandstone CCNBD samples.....	106

Figure 5.18 Stress distribution around the notch crack; left-hand side is maximum principal stress (MPa) σ_1 (+: tension; -: compression), right-hand side is shear stress $\tau_{\max}$ (a) $\beta = 0^\circ$ (b) $\beta = 28^\circ$ (c) $\beta = 30^\circ$ (d) $\beta = 33^\circ$ (e) $\beta = 45^\circ$ (f) $\beta = 70^\circ$	110
Figure 5.19 Line plot post-process option of FRANC2D for stress distribution analysis around notch crack	111
Figure 5.20 Normal stress (σ_n) distribution around the upper face of a notch crack (Tensile: +; Compressive: -)	112
Figure 5.21 Normal stress (σ_n) distribution around the lower face of a notch crack (Tensile: +; Compressive: -)	112
Figure 5.22 Normal stress (σ_n) distribution at the tip of a notch crack (Tensile: +; Compressive: -)	113
Figure 5.23 Shear stress (τ) distribution around the upper face of a notch crack	114
Figure 5.24 Shear stress (τ) distribution around the lower face of a notch crack	114
Figure 5.25 Shear stress (τ) distribution at the tip of a notch crack	115
Figure 5.26 σ_{xx} distribution around the upper face of a notch crack (Tensile: +; Compressive: -) ..	115
Figure 5.27 Simulated original and deformed notch cracks with various inclination angles (a) $\beta = 0^\circ$ (b) $\beta = 28^\circ$ (c) $\beta = 30^\circ$ (d) $\beta = 33^\circ$ (e) $\beta = 45^\circ$ (f) $\beta = 70^\circ$	116
Figure 5.28 Mode I and mode II stress intensity factor values for a crack tip inclined at various angles	117
Figure 5.29 Variation of crack initiation angle (θ) with crack inclination angle (β)	121
Figure 6.1 Sinusoidal cyclic loading	124
Figure 6.2 S—N curve for indirect tensile strength of Brisbane tuff	127
Figure 6.3 Load—number of cycles graph illustrating increasing cyclic loading	127
Figure 6.4 Increasing cyclic loading with (a) same valley and (b) same mean values	128
Figure 6.5 Comparison of load—strain curves of Brazilian disc specimens tested under static and Type I cyclic loading	131
Figure 6.6 Tested Brazilian disc specimens under (a) static loading (b), (c) cyclic loading	132
Figure 6.7 Vertical and horizontal diametral strain reading	134
Figure 6.8 Type I diametral cyclic loading; amplitude = 2.3 kN (10% SUL)	135
Figure 6.9 Type II diametral cyclic loading; amplitude = 2.3 kN (10% SUL)	136
Figure 6.10 Type I diametral cyclic loading; amplitude = 4.6 kN (20% SUL)	137
Figure 6.11 Type II diametral cyclic loading; amplitude = 4.6 kN (20% SUL)	138
Figure 6.12 Type I diametral cyclic loading; amplitude = 6.9 kN (30% SUL)	139
Figure 6.13 Type II diametral cyclic loading; amplitude = 6.9 kN (30% SUL)	140

Figure 6.14 Type I diametral cyclic loading; amplitude = 9.2 kN (40% SUL).....	141
Figure 6.15 Type II diametral cyclic loading; amplitude = 9.2 kN (40% SUL)	142
Figure 6.16 Load–strain curves of Brazilian disc specimens under static and (a) Type I (b) Type II cyclic loading	144
Figure 6.17 Load–strain plots of four cycles at the beginning of test.....	145
Figure 6.18 Load–strain plots of last four cycles up to failure	145
Figure 6.19 Damage and small particles in cement of failed Brisbane tuff disc specimen tested under cyclic loading.....	146
Figure 6.20 Damage and fracturing in cement of failed Brisbane tuff disc specimens tested under monotonic loading	147
Figure 6.21 Typical transcrystalline cracks (a-b) and intracrystalline cracks (c-d) on failure surfaces of Brisbane tuff disc specimens tested under monotonic loading.....	148
Figure 6.22 Scanning electron microscope images of the fractured surface of Brisbane tuff disc specimens (a) loosened grains and typical intercrystalline cracks around grains (b) pulled out grains from cement of tuff	149
Figure 6.23 Scanning electron microscopic images of the fractured surface of Brisbane tuff disc specimens.....	150
Figure 7.1 Sinusoidal cyclic loading.....	153
Figure 7.2 S–N curve for CCNBD specimens.....	155
Figure 7.3 Load–number of cycles graph illustrating dynamic cyclic loading.....	155
Figure 7.4 Dynamic loading cycles (a) with same increasing mean values (b) with same increasing valley values.....	157
Figure 7.5 Comparison of load–CMOD curves of CCNBD specimens tested under static and Type I cyclic loading.....	159
Figure 7.6 Failed specimens and damaged zone in front of the chevron tip (a) under static loading (b) under cyclic loading	160
Figure 7.7 Load–CMOD plots of Type I cyclic loading with amplitude = 10% SUL (0.45 kN)....	162
Figure 7.8 Load–CMOD plots of Type I cyclic loading with amplitude = 20% SUL (0.9 kN).....	163
Figure 7.9 Load–CMOD plots of Type I cyclic loading with amplitude = 30% SUL (1.35 kN)....	164
Figure 7.10 Load–CMOD plots of Type I cyclic loading with amplitude = 40% SUL (1.8 kN)....	165
Figure 7.11 Diametral axial displacement and CMOD versus number of cycles plots; Type I cyclic loading tests with amplitude 10% SUL (0.45 kN).....	167
Figure 7.12 Diametral axial displacement and CMOD versus number of cycles plots; Type I cyclic loading tests with amplitude 20% SUL (0.9 kN).....	168

Figure 7.13 Diametral axial displacement and CMOD versus number of cycles plots; Type I cyclic loading tests with amplitude 30% SUL (1.35 kN)	169
Figure 7.14 Diametral axial displacement and CMOD versus number of cycles plots; Type I cyclic loading tests with amplitude 40% SUL (1.8 kN)	170
Figure 7.15 Comparison of load–CMOD curves of CCNBD specimens tested under static and Type II cyclic loading	173
Figure 7.16 Failed specimens and damaged zone in front of the chevron tip (a) under static loading and (b) under Type II cyclic loading.....	174
Figure 7.17 Load–CMOD plots of Type II cyclic loading with amplitude = 10% SUL (0.45 kN). 175	
Figure 7.18 Load–CMOD plots of Type II cyclic loading with amplitude = 20% SUL (0.9 kN)... 176	
Figure 7.19 Load–CMOD plots of Type II cyclic loading with amplitude = 30% SUL (1.35 kN). 177	
Figure 7.20 Load–CMOD plots of Type II cyclic loading with amplitude = 40% SUL (1.8 kN)... 178	
Figure 7.21 Diametral axial displacement and CMOD versus number of cycles plots; Type II cyclic loading tests with amplitude 10% SUL (0.45 kN)	180
Figure 7.22 Diametral axial displacement and CMOD versus number of cycles plots; Type II cyclic loading tests with amplitude 20% SUL (0.9 kN)	181
Figure 7.23 Diametral axial displacement and CMOD versus number of cycles plots; Type II cyclic loading tests with amplitude 30% SUL (1.35 kN)	182
Figure 7.24 Diametral axial displacement and CMOD versus number of cycles plots; Type II cyclic loading tests with amplitude 40% SUL (1.35 kN)	183
Figure 7.25 Diametral axial displacement and CMOD after the start of nonlinear deformation in front of the chevron crack tip	184
Figure 7.26 CCNBD sample with an inclined crack under diametral compression	185
Figure 7.27 Wing crack initiation positions.....	187
Figure 7.28 $\beta = 30^\circ$ inclined notch and newly formed crack patterns on CCNBD specimens tested under diametral compressive cyclic loading	188
Figure 7.29 Behaviour of $\beta = 30^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 0.45 kN amplitude	190
Figure 7.30 Behaviour of $\beta = 30^\circ$ inclined notch crack in a CCNBD specimen tested under diametral compressive cyclic loading with 0.45 kN amplitude	192
Figure 7.31 Behaviour of $\beta = 30^\circ$ inclined notch crack in a CCNBD specimen tested under diametral compressive cyclic loading with 0.9 kN amplitude	194
Figure 7.32 Behaviour of $\beta = 30^\circ$ inclined notch crack in a CCNBD specimen tested under diametral compressive cyclic loading with 1.35 kN amplitude	195

Figure 7.33 $\beta = 45^\circ$ inclined notch and newly formed crack patterns on CCNBD specimens tested under diametral compressive cyclic loading	196
Figure 7.34 Behaviour of $\beta = 45^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 0.45 kN amplitude	198
Figure 7.35 Behaviour of $\beta = 45^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 0.9 kN amplitude	199
Figure 7.36 Behaviour of $\beta = 45^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 1.35 kN amplitude	200
Figure 7.37 $\beta = 60^\circ$ inclined notch and newly formed crack patterns on CCNBD specimens tested under diametral compressive cyclic loading	201
Figure 7.38 Behaviour of $\beta = 60^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 0.45 kN amplitude	203
Figure 7.39 Behaviour of $\beta = 60^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 0.9 kN amplitude	205
Figure 7.40 Behaviour of $\beta = 60^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 1.35 kN amplitude	206
Figure 7.41 Tension-softening curve	208
Figure 7.42 Debris and small particles in front of the tip of the chevron notch as a result of cyclic loading.....	211
Figure 7.43 Fatigue damage in cement of Brisbane tuff resulting from decohesion of microscale quartz minerals.....	213
Figure 7.44 Fatigue striations in Brisbane tuff cement.....	213
Figure 7.45 Grain decohesion and loosened grains (a), (b), (c), (d) and frictional scratches with high magnification ($\times 1000$) at the surface of fatigue crack	216
Figure 7.46 Fatigue striations in damaged grains by both intragranular and transgranular cracks .	217
Figure 7.47 Cracks around mineral.....	217
Figure 7.48 Damage in cement, and cleavage cracks under monotonic loading	219
Figure 7.49 Damaged grains by transgranular cracking under monotonic loading.....	219
Figure 7.50 Comparison of notch crack deformation under monotonic and cyclic loading.....	220
Figure 7.51 Wedge formation in front of the tip of notch crack after wing and fatigue crack initiation	221
Figure 7.52 SEM image of fatigue crack near the tip of $\beta = 45^\circ$ inclined chevron notch crack.....	222
Figure 7.53 Damage to cement at the surface of fatigue crack.....	222

Figure 7.54 Damaged and loosened grain as a result of transgranular cracking (a, left) and damage to cement because of microscale quartz decohesion (b, right) 222



List of Abbreviations

BSE	Backscattered Electrons
BTS	Brazilian Indirect Tensile Strength
CCNBD	Cracked Chevron Notched Brazilian Disc
CMOD	Crack Mouth Opening Displacement
CSTBD	Chevron Straight Through Brazilian Disc
DAQ	Data Acquisition
DTS	Direct Tensile Strength
EDAP	Extensometer Data Acquisition Program
EDS	Energy Dispersive X-ray Spectrometer
FEM	Finite Element Method
FPZ	Fracture Process Zone
ISRM	International Society of Rock Mechanics
ITS	Indirect Tensile Strength
LEFM	Linear Elastic Fracture Mechanics
LM	Light Microscope
LV	Low Vacuum
NEFM	Nonlinear Elastic Fracture Mechanics
NI	National Instruments
ODC	Oscillating Disc Cutting
PTS-Test	Punch-Through Shear Test
SAX	Single Axis MAX
SCB	Semi-Circular Bend
SE	Secondary Electrons
SEM	Scanning Electron Microscope
SR	Short Rod
STBD	Straight-Through Brazilian Disc
STCA	Straight-Through Crack Assumption
SUL	Static Ultimate Load
UCS	Uniaxial Compressive Strength
UCS	unconfined compressive strength
UTS	Uniaxial Tensile Strength
UTW	Ultra Thin Window

CHAPTER 1

INTRODUCTION

1.1. General

Techniques for underground openings, supports, design of rock pillars, drilling, blasting, haul roads, etc. are based on an understanding of the mechanical behaviour of rocks under various loading conditions. Rock materials are discontinuous at all scales. At the microscale, defects causing stress concentrations are microcracks, grain boundaries, pores, bedding planes, etc., while at the macroscale geologic fractures are referred to as joints (opening) and faults (shearing) based on their genesis. Brittle failure is the product of the creation, growth and accumulation of micro- and macrocracks. Numerous experimental and theoretical efforts have been devoted to the understanding of crack initiation, propagation and coalescence in brittle materials. Under tension, these three processes take place almost simultaneously in brittle rocks. However, the failure process is more complex under compression. Under both kinds of loading, rupture (failure) of the material results primarily from stable and unstable fracture propagation and crack coalescence rather than directly from fracture initiation. In stable fracture propagation, crack extension is a function of loading and can be controlled accordingly. In unstable fracture propagation, crack extension is governed by additional factors beyond loading and thus becomes uncontrollable.

Rock is a brittle material, and all brittle materials are much weaker in tension than in compression. Focusing on the tensile strength properties of rocks under different loading conditions has become more popular in the last few decades. Several methods have been proposed by the International Society of Rock Mechanics (ISRM) in order to determine indirect tensile strength and mode I fracture toughness values for rocks under monotonic loading. These methods could be classified in terms of their loading conditions. The existing theories and models proposed for deformation and failure of brittle materials under indirect tensile monotonic loading can be attributed to the development of stress-induced microfractures under indirect tensile cyclic loading. The practical application of these theories (i.e., Griffith crack theory, stress corrosion, etc) primarily involves the determination of stress thresholds at which a crack will initiate and propagate. Thus, concentrating on failure due to tensile cracks has allowed for the correlation of the microfracturing process with the progressive failure of a rock material under cyclic loading. Thus, laboratory experiments can provide us with fundamental data to develop a conceptual model of the failure process under

various loading conditions, such as monotonic and cyclic loading, and help us to consider the coupled factors that affect the behaviour of rock, including pre-existing fractures.

1.2 Scope of Research

The work described in this thesis involves two fundamental research approaches: (1) investigating the fracturing of rocks under both monotonic and cyclic indirect tensile loading and (2) investigating the effect of cyclic loading on tensile strength properties of rocks, such as indirect tensile strength and fracture toughness. The effect of cyclic loading on tensile strength properties of rocks is a very new subject in rock mechanics and rock fracture mechanics. In contrast, there have been critical discussions on some proposed international standard methods for determination of indirect tensile strength and fracture toughness of rocks under monotonic loading ever since they were proposed. Thus, detailed investigation and criticism of the accuracy of current international standards to determine indirect tensile strength and fracture toughness of rocks under monotonic loading was done at the beginning of this research. The outcomes of the monotonic loading tests were then extended in a second series of cyclic loading tests to investigate the effect of rock fatigue on indirect tensile strength and fracture toughness of rocks by comparing them to the monotonic test results.

Cyclic fatigue in structural materials such as metals, concrete and soil has been thoroughly investigated over the last few decades. Rock fatigue, however, has received little attention. Faults, joints, bedding planes, tunnel walls, excavation roofs and ribs, bridge abutments and dam and road foundations are some of the natural and man-made rock structures that can be weakened by repetitive loading. A review of the literature has shown that there has been limited progress made on the description of rock fatigue. Thus, the available data are not sufficient to allow reliable prediction of rock strength degradation due to cyclic loading, and are insufficient for the solution of tunnel design problems, underground repositories, rock burst problems, etc. Hence, more investigation is necessary to clarify the mechanisms of damage/fracturing of rocks under cyclic loading. There is also very limited research on the effects of fatigue on tensile strength properties of rocks under cyclic loading (as opposed to dynamic loading, such as explosive loads).

In brief, this research involves the following fundamental steps:

- Determination of Uniaxial Compressive Strength (UCS) of Brisbane Tuff
- Determination of Uniaxial Tensile Strength (UTS) of Brisbane Tuff by using indirect and direct tension tests

- Determination of Young's Modulus, Poisson's ratio and Shore Scleroscope Hardness of Brisbane Tuff
- Determination of Mode I fracture toughness of Brisbane Tuff by using different ISRM-suggested methods and their comparison
- Effect of angle of loading arc on the indirect tensile strength of the rock
- Effect of notch inclination angle on the fracturing of rocks
- Determination of the S–N curve for the first time in the literature to obtain the fatigue limit of the indirect Brazilian tensile strength of the Brisbane Tuff
- Determination of the S–N curve for the first time in the literature to obtain the fatigue limit of the fracture toughness of the Brisbane Tuff using a Cracked Chevron Notched Brazilian Disc (CCNBD)
- Investigations of the effect of cyclic loading with various amplitudes on the Mode I fracture toughness of Brisbane tuff
- Investigations of the effect of cyclic loading with various amplitudes on the mixed-mode I–II fracturing of Brisbane tuff
- Investigations of the effect of cyclic loading with various amplitudes on the indirect tensile strength of Brisbane tuff by using Brazilian disc specimen
- Investigations of the fatigue damage mechanism using low-vacuum Scanning Electron Microscope (SEM) imaging for direct observation of fracture surfaces

1.3 Thesis Layout

This thesis is organised into nine (9) chapters, and begins, in Chapter 1, with a brief introduction to rock fracture mechanics, a statement of the problem, the thesis objective and the methodology used in the research. Chapter 2 covers the history and the terminology of rock mechanics and rock fracture mechanics. Chapter 3 continues with a description of the experimental techniques and equipment used. A comparison between experimental results of direct and indirect tensile tests carried out on three rock types, and analytical and numerical analysis is covered in Chapter 4. Two test methods suggested by the ISRM, the Short Rod (SR) and the CCNBD tests, were used to determine the mode I fracture toughness of Brisbane tuff, and a comprehensive comparison between the two sets of results obtained is made in Chapter 5. Mixed-mode (I–II) fracture propagation of cracks in rocks is also considered, based on experimental results and numerical analysis. An analysis of stress distribution, discrete crack propagation, and crack initiation and propagation angles is carried out using a finite element program, FRANC2D (Fracture Analysis

Code). Two different types of cyclic loading are described in Chapters 6 and 7: (1) cyclic loading with constant mean level and constant amplitude, termed sinusoidal cyclic loading, and (2) cyclic loading with increasing mean level and constant amplitude, termed increasing cyclic loading. In Chapter 6, results of laboratory experiments on the stress–strain characteristics of Brisbane tuff Brazilian disc specimens under diametral compressive cyclic loading are presented. The degradation of the indirect tensile strength of rocks due to rock fatigue is also presented in Chapter 6. In Chapter 7, the degradation of rock fracture toughness as a result of rock fatigue is presented. SEM was used to analyse damage due to rock fatigue. Chapter 8 discusses the results obtained. In Chapter 9, conclusions drawn from this research and recommendations for further research are presented.



CHAPTER 2

DEFORMATION AND FRACTURING OF BRITTLE ROCKS

There is no doubt that an understanding of the mechanics and mechanisms of brittle rock fractures is a key element in the solution of many engineering problems that involve rock structures. The theories, criteria and testing methods in rock mechanics or rock fracture mechanics were originally developed with man-made materials in mind and, consequently, they needed to be adapted to rock materials both flexibly and critically. The introduction of the fracture mechanics approach to rock mechanics has led to the development of rock fracture mechanics, which mainly refers to the discrete initiation and propagation of an individual crack or cracks in rock subjected to a particular stress field. Deformation in brittle rock materials is a complex problem, and often requires advanced techniques for the prediction of fracture initiation, stable/unstable fracture propagation and geometry.

2.1 Introduction

As a subclass of solids, rocks and rock like materials are divided into two classes, brittle or ductile, depending on their deformation response to an applied load; the more irreversible plastic deformation occurs, the more ductile, or less brittle, the solid material is in relation to its load-deformation behaviour. The load deformation curve obtained in a laboratory test can be transformed into a stress–strain function. For instance, the stress–strain function of a typical ductile material first reaches a peak load, or so-called initial yield point, followed by a long plastic strain, which is a clear indication of material failure by a microstructural dislocation mechanism, rather than by propagation of inherent tiny cracks and flaws, as occurs in most brittle materials. Brittle materials normally produce very small regions of ductile behaviour before their final fracture, while ductile materials, as well as highly confined rocks, can develop very large regions of ductile behaviour.

Rock brittleness is an important mechanical property in rock mechanics, although it seems there are controversial disagreements amongst different scientists as to both its definition and measurement. For instance, Ramsay (1967) defines brittleness as ‘when the internal cohesion of rock materials which are deforming in their elastic range is broken, the rocks are said to be brittle’, and Coates (1966) defines brittleness measurement as ‘the ratio of the reversible strain to that of the total strain at the point of failure’. Irrespective of definitions, there is a strong correlation between rock brittleness and opening, or progression or propagation of pre-existing cracks. Rock fracture strength

or fracture toughness is a measure of the behaviour of these natural flaws. Typical brittle failure in laboratory tests of rock cutting is characterised by chip formation and separation due to the combined action of shear and tensile fracturing initiated in a crushing zone near the tooth tip that propagates into the intact rock (Evans, 1961; Goodrich, 1956; Nishimatsu, 1972).

Rock masses normally consist of blocks of intact rocks separated by joints, faults or interfaces. Even intact rocks are categorised as an inhomogeneous, discontinuous material, on both small and large scales, because they contain cracks, voids and pores, normally induced by historical thermal, mechanical and/or chemical actions and reactions. Intact rocks can be laminated and anisotropic and form distinct layers of bedding planes. When rocks are loaded, an internal stress flow is generated that travels through the clear paths between several cracks with the potential for jamming around crack tips, causing material failure.

Cracks are stress concentrators in loaded rocks and are the main sources of further fracture and damage by propagation and coalescence with other cracks. Study of brittle fracture and its relation to material damage, fatigue and strength is central in many rock engineering disciplines including rock fracture and damage mechanics, rock cutting, drilling and blasting. Research into the understanding of rock crack, fracture and fatigue behaviour under various loading conditions such as static, cyclic and impact loading, has attracted many investigators in recent years (Celestino *et al.*, 1995; Ingraffea, 1981; Petros and Bagde, 2003; Tao & Mo, 1990).

The fatigue life of solids depends on both number of loading cycles and loading amplitude. The loading amplitude is generally less than the peak static load causing material failure. There is, however, little published research work on cyclic and fatigue behaviour of rocks (Attawel & Farmer, 1973; Bagde and Pedros, 2005; Gattelier *et al.*, 2002; Haimson, 1978; Tao & Mo, 1990). There are several potential mechanisms for fatigue failure of rocks, such as along joints, faults and bedding planes, or in microcracks or flaws caused, e.g., by vibrations from earthquakes or blast loading.

Another important property of rock in relation to its brittleness is its low ratio of tensile to compressive strength, again due to the presence of microfractures. Rocks could have tensile strengths more than 10 times less than their measured compressive strengths. Therefore, damage, fracture, fatigue and failure analyses of rocks are better explained in terms of their weaker tensile strength and fracture toughness, rather than the higher Unconfined Compressive Strength (UCS) of

rocks. This chapter discusses various aspects of brittle fracturing of rocks in terms of pre-existing cracks and fracture toughness, as measures of rock tensile strength.

2.2 Theories of Brittle Fracture

When a rock sample is under an increasing load, a dominant internal stress is also increased until it reaches a critical stress level, the so-called rock strength. Similarly, when a cracked rock sample is under an increasing load with the effect of opening or shearing the crack, a dominant internal stress across the crack tip forms, causing an increase in the crack length until it reaches an ultimate crack resistance state, called fracture toughness. Measured values of fracture toughness of a cracked sample and the tensile strength of an uncracked sample of the same material are considered as inherent properties of the material's brittleness, because it fails soon after loading beyond these critical values, with no significant stored plastic strain energy.

Brittle failure is the product of the creation, growth and accumulation of micro- and macrocracks. Inglis (1913) and Griffith (1924) first recognised the significance of pre-existing cracks acting as precursors to failure. Inglis (1913) provided a mathematical solution to the stress magnitudes around an elliptical hole in a stressed thin plate. Based on Inglis's (1913) stress analysis, Griffith (1924) established a relation between the fracture causing stress and the crack size. Brace (1964) and Hoek (1965) modified Griffith's brittle fracture theory (Griffith, 1924) to account for the effects of compressive crack closure and shearing in their fracture mechanics study of hard rock. Their theoretical and experimental studies led to a hypothesis on the mechanism of rock fracture under compressive loading (Bienavski, 1966; Nemat-Nasser, 1995). The general scheme of the brittle fracture process after Irvin's (1960) modification is shown in a block diagram, as Figure 2.1. As shown in the diagram, the process of fracture propagation in rock is characterised by two phases, i.e., stable and unstable crack growth. The transition between these two phases during fracturing or crack growth is determined by the value of a fracture parameter called the 'critical energy release (G)', which will be discussed later through Equations (2.4) and (2.5). Crack propagation is either stable or unstable, depending on loading conditions. It is unstable when loading is increased independent of crack opening deformation and stable otherwise, i.e., when full control of the crack opening deformation, as a function of loading intensity, can be achieved.

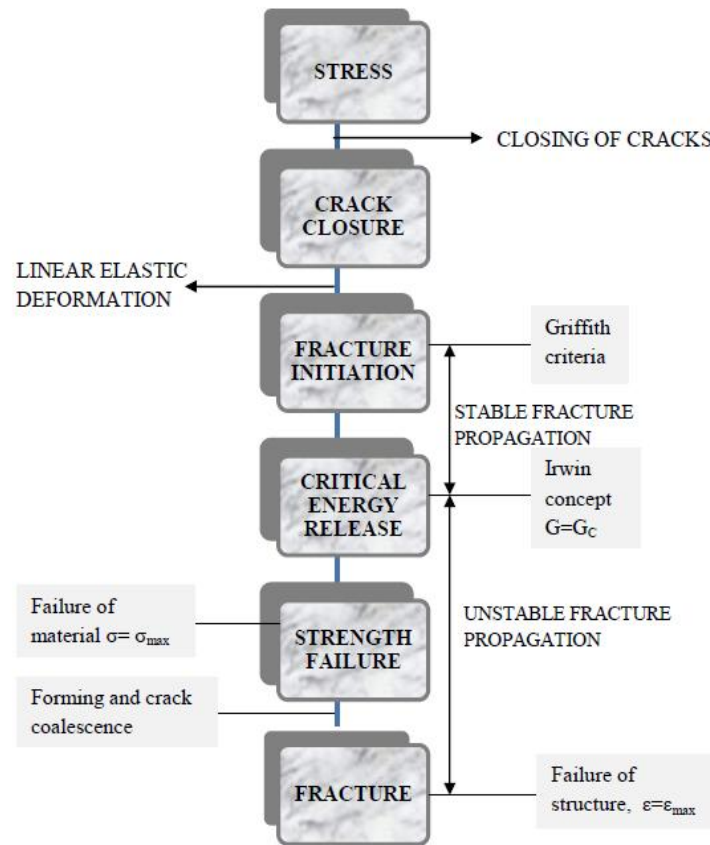


Figure 2.1 Brittle fracture process in a rock crack is either stable or unstable, depending on the value of the crack critical energy release (G)

The original Griffith theory was developed for tensile cracks, i.e., cracks under a tensile load; hence, it was not applicable to compressive cracks, which demand a more rigorous approach. In extending the theory to compressive loading, Griffith resorted to the examination of tensile stress concentrations around the periphery of a single isolated, open, elliptical crack of variable orientation.

2.2.1 Griffith's theory

Griffith (1920) realised the significance of inherent flaws or microcracks in reducing a material's strength and also in measuring its imperfection. Griffith argued that brittle solids fail by incremental propagation of a multitude of randomly oriented, small pre-existing cracks. Griffith cracks are common in rocks that contain both intragranular (outside; in grain boundaries) and intergranular (inside; in grains) microcracks and larger macroscopic or transgranular multiple cracks.

Griffith's theory is also applicable to macroscopic or larger scale cracks including rock joints, faults and bedding planes. One important feature of the Griffith theory is that while the microscopic fracture stress is relatively very large, it applies only to a very small volume of the rock around the crack tip. Griffith's work, which has since become known as the Griffith energy balance approach, and which has served as a foundation for fracture mechanics, deals with the equilibrium state of an elastic, solid body surrounding a crack, deformed by specified surface forces.

Based on his experiments with a plate with a circular hole, Griffith (1920) states that there should be a simple energy balance relation to explain the boundary deformation and work by the applied stresses, the energy release from the strain potential energy of the stressed body, and the expenditure of free surface energy as a result of increasing fracturing surface area for crack extension. In other words, the total energy U of the crack plate is equal to the summation of the boundary applied work, W_L ; the strain potential energy, U_E ; and the free surface energy in the free faces of Griffith's crack, U_S :

$$U = (-W_L + U_E) + U_S = U_{Mech} + U_S = -U_E + U_S \quad 2.1$$

Equation (2.1) is the essence of Griffith's energy balance concept, where the bracket term U_{Mech} , also called the mechanical energy, favours crack extension, while the U_S term opposes it. He then used Inglis's (1913) solution to the problem of a thin elastic plate of unit thickness with a circular hole of radius a , subjected to uniaxial tensile boundary stress (σ) under plane stress conditions, to calculate the elastic strain potential energy U_E as follows:

$$U_E = \pi \sigma^2 a^2 / E \quad 2.2$$

where E is Young's modulus. Another Young's modulus, however, needs to be defined when using different calculation models. The calculation of Young's modulus, E' , is different for plane stress and plane strain conditions: $E' = E$ for plane stress, but $E' = E / (1 - \nu^2)$ for plane strain where ν is Poisson's ratio. Griffith then defined a free surface energy term, γ , which he calculated to be 1.75 J/m² for the glass he used in his experiments and proposed the following relation for the crack surface energy:

$$U_S = 4a\gamma \quad 2.3$$

The equilibrium condition of the total system energy does not require variation in the total energy with crack length; in other words

$$\frac{dU}{da} = -\frac{2\pi\sigma^2 a}{E'} + 4\gamma = 0 \quad 2.4$$

In Equation (2.4), G is the Griffith elastic strain energy release rate at the crack tip, defined as half of the elastic strain energy rate;

$$G = \frac{1}{2} \frac{dU_E}{da} = \frac{\pi\sigma^2 a}{E'} \quad 2.5$$

Griffith also expanded his original formulation to include uniaxial and biaxial loading causing compressive stress fields around an elliptical open crack. He postulated that even when the applied boundary stresses change from tensile to compressive, the local stress at the crack tip causing crack opening would be tensile. Griffith found from his experiments that the applied compressive stress required for crack growth in his experimental glass is eight times that required for tensile loading. In other words, the glass compressive strength was eight times its tensile strength:

$$\sigma_c = 8 \sqrt{\frac{2E\gamma}{\pi a}} \quad 2.6$$

In Equation (2.6) σ_c is the applied compressive stress required for crack propagation and γ is the same free surface energy per unit area of crack surface as discussed earlier. Griffith did not consider the possibility of crack growth under a shear loading effect at the crack tip.

Under compressive loading conditions, a crack may first close and then undergo a frictional shear deformation before starting a tensile opening in a new direction perpendicular to the maximum principal tensile stress direction. In this case, frictional shear stresses caused by the contact normal stress or pressure between the crack faces have to be overcome before any tensile crack initiation or propagation can occur. A tensile crack must have a tensile stress at its crack tip. Therefore, all compressive and shear cracks grow until a re-oriented tensile crack develops.

In the application of Griffith's energy balance theory (Griffith, 1924) to uniaxial tensile strength calculations, it is assumed that the crack extension is in its own plane. However, difficulties arise in

evaluating experimentally the surface energy of a given material when applying Griffith's theory to more complicated stress fields involving compression. In this case, it can no longer be assumed that the crack extends in its own plane but in an orientation with respect to the crack plane (Cook, 1965; Hoek and Bieniawskie, 1965; Jaeger, 1969). To eliminate these problems, Griffith proposed a critical maximum local tensile stress criterion for an open crack extension from the crack tips. The interpretation of this approach is that a crack initiates when tensile stress induced at or near the tip of an open crack exceeds the cohesive strength of the material. In contrast to the energy approach previously presented, this is known as the Griffith fracture stress criterion (Whittaker, 1992).

2.2.2 Modified Griffith fracture stress criterion and linear elastic fracture mechanics

Griffith's fracture theory laid the foundation for a failure criterion for macroscale fracturing of brittle materials. Various modifications to Griffith's theory have been suggested in order to extend its application to other materials, e.g., the work of Irvin (1948), McClintock and Walsh (1962), Cook (1965), and Jaeger (1966). One interesting modification is the failure criterion proposed by McClintock and Walsh (1962):

$$\sigma_1 = \sigma_3 \left[1 + \frac{2f}{\sqrt{1 + f^2 - f}} \right] + \sigma_c \quad 2.7$$

where σ_1 and σ_3 are the principal stresses, and f is the coefficient of friction between the crack faces. In order to compare the applicability of the modified theory to the prediction of rock fracture behaviour at a macroscale level, Hoek and Bieniawski (1965) suggested the following relation for calculation of the coefficient of friction (f) between crack surfaces when the normal stress acting across the crack is compressive:

$$\tan(2\sigma_c) = \frac{1}{f} \quad 2.8$$

Although Griffith's theory has its limitations, i.e., crack extension is accepted as in its own plane, it formed a milestone in the history of research into brittle fracture. Griffith's greatest contribution is the development of a relationship between the fracture stress and crack size, which inspired and stimulated the progress of modern fracture mechanics of solids. Modern fracture mechanics is a relatively new branch of material science that seeks to quantify the critical combinations of stress and crack size for crack extension in a general nonhomogeneous material with existing or stress-

induced defects in the form of voids, microcracks and weak interfaces (Knott, 1973; Whittaker *et al.*, 1992). Engineering fracture mechanics accounts for the inadequacies of conventional design methods by considering the effect of discontinuities/cracks, which inherently exist in any brittle material, since fracture mechanics was originally developed with man-made materials in mind.

In Linear Elastic Fracture Mechanics (LEFM), the crack tip elastic stresses, strains and displacements are characterised by a single parameter—namely the stress intensity factor, K . The quantity K is essentially a measure of stress intensity at the tip of a crack, which characterises crack tip stresses, strains and displacements in the crack tip inelastic zone, provided the so-called process zone is small compared to the other characteristic dimensions of the structure containing the crack. Based on the principles of LEFM, a number of branches of fracture mechanics have been identified and developed in order to meet practical demands, examples of which are dynamic fracture mechanics and nonelastic fracture mechanics.

2.3 Initiation and Propagation of Cracks in Brittle Rocks

Griffith's theory established the fact that material strength is a strong function of both crack length and crack orientation, particularly in relation to crack tip principal stresses; the longer a crack, the weaker the material containing the crack. Hence, there is a critical crack length for material fracture stability analysis. Fredrich *et al.* (1990) observed that an increase in crack density (in fine-grained materials) can cause an increase of material heterogeneity and anisotropy of the local stress field. Inglis (1913) first demonstrated that stresses forming on the boundary of an ellipse will vary depending on the orientation of an elliptical crack with respect to the direction and sign (compressive or tensile) of the load. He confirmed this important result that a maximum tensile stress many times larger than the applied loading stress can be generated at the boundary of the ellipse modelling crack geometry.

However, pre-existing cracks in rock material and discontinuities in rock masses are seldom subjected to tensile loading, but rather to compressive, shear or mixed modes of loading. Tensile stresses around the ellipse boundary can be generated even under compressive loading. The tensile mechanism of crack propagation in itself is not sufficient for fracturing of rocks (Lajtai, 1971). Because failure in rocks usually involves faulting (shear fracture), another failure mechanism, which is probably controlled by shear stresses, should become active at some later stage of the fracture process. Lajtai (1971) showed that the highest tensile stress concentration on an elliptical

boundary was inclined to the major principal stress at approximately 30° , under uniaxial compressive loading conditions.

Relating to the field of rock mechanics, a number of experimental studies have been carried out to investigate the mechanism of crack initiation, propagation and interaction under various stress states (Hallbauer *et al.*, 1973; Pang & Johnson, 1974; Tappin & Brace, 1976). At the same time, many mathematical models have been developed to explain and predict the processes of crack growth, interaction and rock failure (Brace & Bombalakis, 1963; Day & Wang, 1981; Hori & Nemat-Nasser, 1985; Kemeny, 1991; Kemeny & Cook, 1987; Nemat-Nasser & Horii, 1982). Brace and Bombalakis (1963) proposed a novel sliding crack model, as a tool to study inelastic dilation behaviour of rock materials under compressive loading. It contains an initial crack oriented at an angle with respect to the compressive load, and a pair of curved tensile (wing) cracks, oriented at an angle to the initial crack. According to their model, tensile cracks are caused by the sliding of the initial crack under compressive and/or shear loading. An exact analytical solution of the stress field for the sliding crack model has been given by Horii and Nemat-Nasser (1985).

Because the state of stress in the vicinity of the crack tip is altered significantly once a crack has opened, prediction of the direction of crack propagation and orientation of a fracture in brittle rock material becomes crucial. LEFM states that a crack will propagate when its stress intensity reaches a critical value, K_{IC} . The stress intensity factor depends on fracture displacement modes and crack geometry. A crack can deform in three basic modes, tensile, shearing and tearing, or the so-called mode I, mode II and mode III, respectively; see Fig. 2.1. This classification of fracturing is based on crack surface displacement or crack tip loading (Lawn, 1993; Whittaker *et al.*, 1992).

In Mode I, the crack tip is subjected to a normal stress, so that the two crack surfaces open up and separate from each other normal to the crack extension direction. Mode II corresponds to edge sliding or in-plane shearing of the crack surfaces, where the crack tip is subjected to an in-plane shear stress so that one crack surface slides relative to the other parallel to the crack extension direction. In Mode III or tearing mode, the crack tip is subjected to an out-of-plane shear stress whereby the crack surfaces move relative to each other perpendicular to the crack extension direction (Figure 2.2).

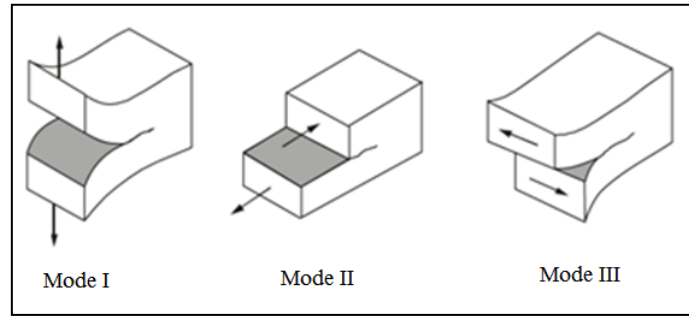


Figure 2.2 Three fracture modes and corresponding crack surface displacements

Crack propagation behaviour changes with orientation and geometry. The smooth mathematical geometry of an ellipse, in various forms of orientation and aspect ratio (ratio of ellipse axis lengths), has been used to model cracks during the development of a series of models and failure criteria of fracture mechanics (Figure 2.3; e.g., Brace & Bombolakis, 1963; Carpinteri *et al.*, 1996, Hoek & Bienawskie, 1965; Kemeny & Tang, 1990; Lajtai, 1971; Peng & Johnson, 1972; Rossmanith, 1983; Wawersik & Brace, 1971).

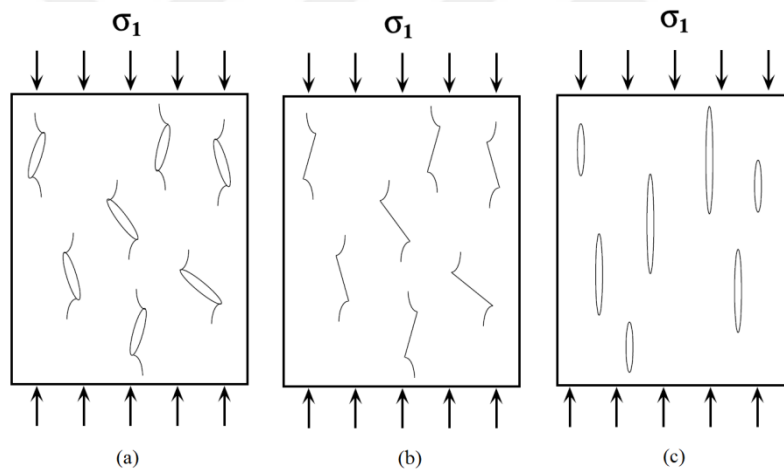


Figure 2.3 Crack geometry and inclination used in various crack modelling methods (a) inclined ellipse cracks (b) inclined cracks with zero width (c) axial ellipse cracks

Cracks propagate differently in heterogeneous materials, taking different names and definitions. Direct observations of microcracks by SEM, optical microscope and petrographic methods have identified four types of cracks in rocks (Eberdhard, 1998; Kranz, 1983):

- (1) grain boundary cracks;
- (2) intragranular cracks, which lie totally within the grain;
- (3) intergranular cracks, which extend from a grain boundary crossing into another grain; and
- (4) multigranular cracks, which cross several grains and grain boundaries.

The mechanism of crack initiation is simply tensile, while the mechanism of failure is complex and requires the coalescence of several microcracks. Griffith (1924) put forward the important notion that crack initiation, even under compressive loading, results from local tensile stresses at the crack tip. Crack initiation, propagation and specimen failure do not occur at the same time; rather each separately forms a particular fracture process. Cracks can also be stopped from further growth by other mechanisms such as arrestment and overlapping. Further, crack initiation from a pre-existing crack tip, called a primary crack, does not have to lead to failure of the specimen due to the arrestment of the initial crack at some loading level. With increasing load, secondary cracks can develop and propagate until reaching the specimen boundaries, leading to final failure or acting as a ligament for primary cracks to form the final failure plane.

Natural rock masses, subjected to complex loading conditions, normally contain cracks of different and arbitrary scales, types and orientations. Because of the arbitrary orientation of the cracks with respect to the loading directions, fracturing of brittle rocks occurs under various mixed modes, rather than in a single mode. This is why research into mixed-mode fracturing mechanisms has been so attractive in recent years (e.g., Al-Sheyeh, 2005; Ayetollahi & Aliha, 2007; Backer, 2004; Hasanpour & Choupani, 2009; Rao *et al.*, 2003).

Both modes I and II play interactive roles in a compressively loaded rock specimen. It is generally accepted that obliquely oriented natural microcracks, which can grow in pure tension at an angle with respect to the direction of the axial compressive stress, propagate in the tensile mode I, followed by rapid growth parallel to the axial compressive stress (Bienawskie, 1967; Hoek, 1965; Li *et al.*, 2005; Lin *et al.*, 2009). However, few literature and experimental results exist on the mixed-mode behaviour of various cracks, particularly under compressive loading, e.g., Al-Sheyeh, 2005; Ayetollahi & Aliha, 2007; Barker, 1977; Chang *et al.*, 2002; Chen, 1990; Huang & Wang, 1985).

Applied loading stress translates into a stress intensity factor at the crack tip. The equations given by LEFM to calculate stresses and stress intensity factor for a crack under tensile loading are also applicable for a crack under compressive loading, but use opposite signs, i.e., K_I and K_{II} become negative. A negative K_I indicates compressive stress acting at the crack tip and a negative K_{II} just indicates an opposite direction for the shear stress acting parallel to the crack plane. As shown in Figure 2.4, the inclination angle (θ) of a compressively loaded crack is such that crack extension

occurs parallel to the major principal compressive stress (Figure 2.4a), while it is parallel to the minor principal tensile stress in the case of a crack under a tensile load (Figure 2.4b).

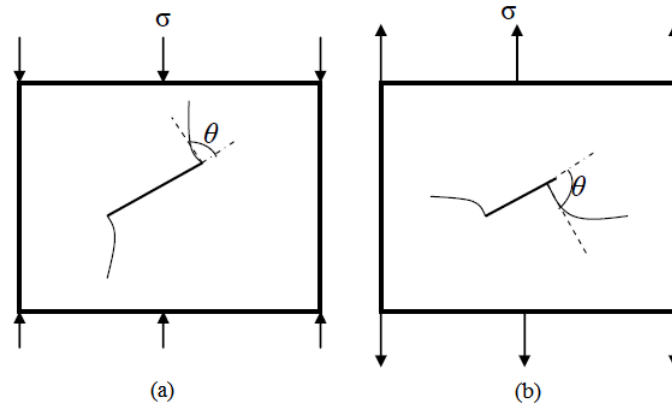


Figure 2.4 Crack propagation paths under compression (a) and tension (b)

Various criteria have been suggested to explain crack initiation and propagation under mixed-mode I–II loading. The related fracturing stresses for mode I, mode II and mixed-mode I–II loading conditions in the case of compression can be found similar to those for the case of tension by using the three fracture criteria used in fracture mechanics (Erdogan & Sih, 1976; Hussain & Pu, 1974; Sih, 1974). In contrast, some researchers have provided a closed-form solution for stress distribution in cracked and uncracked disc specimens under diametral compression (Atkinson *et al.*, 1982; Avaji and Sato, 1978; Shetty *et al.*, 1985). Shetty *et al.* (1985) first used a CSNBD specimen to calculate both mode I and mode II fracture toughness of ceramics. Moreover, Atkinson *et al.* (1982) and Avaji and Sato (1978) developed dimensionless mode I stress intensity factor (NI) and dimensionless mode II stress intensity factor (NII) solutions depending on the dimensionless notch length α (a/R) and the notch inclination angle (β) with respect to the loading direction.

2.4 Tensile Strength of Rocks

As brittle materials, rocks exhibit different behaviours under tensile and compressive loading. The tensile strength of rock is orders of magnitude less than its compressive strength. However, direct rock tensile tests are not common because they are more expensive and more difficult than indirect testing methods. In 1978, the indirect Brazilian test was officially proposed by the ISRM as a method for determining the tensile strength of rock materials (ISRM, 1978). The Brazilian test, or splitting tension test, is performed by applying a concentrated compressive load across the diameter of a disc specimen. The Brazilian test is also a suggested method for determining the tensile

strength of hardened concrete specimens. Hondros (1959) developed an approach to measure the elastic modulus and Poisson's ratio of rock samples from the Brazilian disc test. He also formulated a complete stress solution for the case of a radial load distributed over a finite circular arc of the disc. However, the Brazilian test has been criticised by a few rock mechanists since its introduction. Fairhurst (1964) first discussed the important issue of the validity of the Brazilian test. He stated that 'failure may occur away from the centre of the test disc for small angles of loading contact area' and also argued that the tensile strength calculated from a Brazilian test is lower than the true value of the tensile strength.

Although the Brazilian test has been studied extensively, both experimentally and theoretically, relatively little attention has been directed towards researching the validity of the test. Several key questions remain unresolved, for example, how can the classical idea of crack initiation at the centre of the Brazilian disc specimen (beneath the concentrated load) be guaranteed? How accurate and reliable is the calculated tensile strength in relation to its true tensile strength? How accurate and reliable is the load distribution function across the Brazilian disc diameter? What closed-form solutions should be used or developed to provide complete stress and strain fields within the disc, which are consistent with the true loading boundary conditions and material heterogeneity and anisotropy, particularly when the load is applied as a pressure acting normal to, and uniformly across, an arc of finite length? Hudson *et al.* (1972) found that the tensile strength of rock varies considerably and depends on the following factors: method of testing; rock heterogeneity and anisotropy and loading boundary conditions, i.e., contact conditions between the specimen and the steel platens of the testing machine (rough or smooth contact). He observed that, 'In the Brazilian test, it was found that failure always initiated directly under the loading points if flat steel platens were used to load the specimen', which actually invalidates the test for the determination of tensile strength. Wang *et al.* (2004) used specimens with two parallel flat, square ends at the loading points to prevent local crack initiation at the loading points. However, they found that the testing requirements of square specimens in which both ends are completely flat and parallel, are highly critical for a successful test.

Assuming two concentrated, diametrical line loads, applied over an infinitesimally small width, the classical elastic theory calculates the tensile strength of a rock disc specimen, σ_t , from the following equation:

$$\sigma_t = \frac{2P}{\pi dt} \quad 2.9$$

where P is the measured Brazilian failure load, and d and t are the diameter and thickness of the rock disc specimen, respectively. However, the actual loads are not concentrated but are distributed over two opposite finite arcs of the disc, which would lead to less stress concentration in the vicinity of the loading boundary (ISRM, 1978).

When a disc is diametrically compressed under a line load P , the stresses at any point in the disc specimen are as follows (Yu *et al.*, 2006):

$$\begin{aligned}\sigma_x &= \frac{2P}{\pi t} \left(\frac{\sin^2 \theta_1 \cos \theta_1}{r_1} + \frac{\sin^2 \theta_2 \cos \theta_2}{r_2} \right) - \frac{2P}{\pi d t} \\ \sigma_y &= \frac{2P}{\pi t} \left(\frac{\cos^3 \theta_1}{r_1} + \frac{\cos^3 \theta_2}{r_2} \right) - \frac{2P}{\pi d t} \\ \tau_x &= \frac{2P}{\pi t} \left(\frac{\cos^2 \theta_1 \sin \theta_1}{r_1} + \frac{\cos^2 \theta_2 \sin \theta_2}{r_2} \right)\end{aligned}\tag{2.10}$$

where, r_1 and r_2 are polar distances. See Fig. 2.5 for descriptions of the other symbols used.

According to Jaeger and Cook (1976), if a circular cylinder of radius r is compressed across its diameter between flat surfaces that apply concentrated loads of W per unit axial length of the cylinder, the stresses on this diameter are:

$$\begin{aligned}\sigma_x &= -\frac{W}{\pi r} \\ \sigma_y &= \frac{W(3r^2 + y^2)}{\pi r(r^2 - y^2)}\end{aligned}\tag{2.11}$$

By symmetry, these are principal stresses and the major and minor principal stresses are along and across the y -diameter, respectively. If the load is applied to the circumference of the cylinder as a pressure p distributed over an arc 2α by using shaped platens, so that $W = 2p\alpha r$, then equal biaxial compression exists near the contacts with a value of p (Jaeger & Cook, 1976).

A distributed load applied to a disc under diametral compression is more difficult to analyse than that of a concentrated load. Hondros (1959) analysed the Brazilian test for the case of a thin disc loaded by a uniform pressure, applied radially over a short strip of the circumference at each end of the disc. He obtained the full-field stresses using the series expansion technique as follows:

$$\begin{aligned}
\sigma_r &= -\frac{2p}{\pi} \left\{ \alpha + \sum_{n=1}^{n=\infty} \left[1^\infty \left(1^\infty \frac{1}{n} \right) \left(\frac{r}{R} \right)^2 \right] \left(\frac{r}{R} \right)^{2n-2} \sin 2n\alpha \cos 2n\theta \right\} \\
\sigma_\theta &= -\frac{2p}{\pi} \left\{ \alpha - \sum_{n=1}^{n=\infty} \left[1^\infty \left(1^\infty \frac{1}{n} \right) \left(\frac{r}{R} \right)^2 \right] \left(\frac{r}{R} \right)^{2n-2} \sin 2n\alpha \cos 2n\theta \right\} \\
\tau_{r\theta} &= -\frac{2p}{\pi} \left\{ \sum_{n=1}^{n=\infty} \left[1^\infty \left(\frac{r}{R} \right)^2 \right] \left(\frac{r}{R} \right)^{2n-2} \sin 2n\alpha \cos 2n\theta \right\}
\end{aligned} \tag{2.12}$$

where p is applied pressure, R is the radius of the disc, r and θ are the polar coordinates of a point in disc, and α is the half central angle of the applied distributed load (Figure 2.5). It can be seen from Equations 2.12 that the magnitude of α affects the stress distribution within the disc directly.

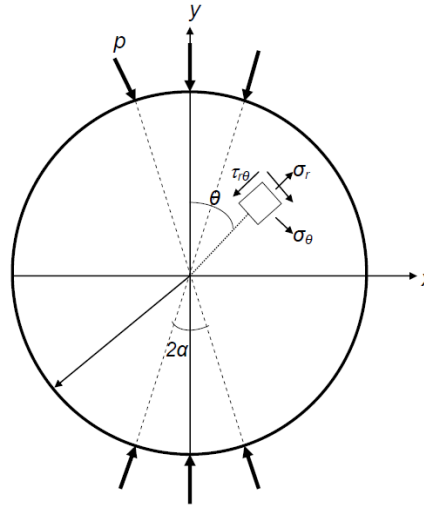


Figure 2.5 A disc specimen subjected to diametric distributed compression

Recent research (Cai & Kaiser, 2004; Lanaro *et al.*, 2009) show that the formula for calculating indirect tensile strength from the failure loads obtained by Brazilian tensile testing might not be applicable for rocks containing minerals with different strengths, flaws and cracks. However, in both numerical and analytical solutions a rock specimen is assumed to be homogenous, isotropic and linear elastic. Because pre-existing or newly initiated flaws and cracks affect the stress distribution in the specimens, such specimens cannot be assumed to comply with this assumption. Further, the closed-form analytical solution assumes that failure occurs in pure tension. Some of the available studies show that failure may occur in both tension and shear (Lanaro *et al.*, 2009; Van De Steen *et al.*, 2005). Thus, the indirect tensile Brazilian disc test needs to be modified by considering the heterogeneity of rocks and by including some intrinsic rock properties such as the frictional coefficient.

2.5 Fracture Toughness of Rocks

In LEFM, the crack tip elastic stresses, strains and displacements are characterised by a single parameter—namely, the stress intensity factor K . The formulation of the stress intensity factor is:

$$K_{(I,II,III)C} = \sigma_A \sqrt{\pi a} = \sigma_{ij} \sqrt{2\pi r} \quad 2.13$$

where σ_A is applied stress, σ_{ij} is the stress tensor in Cartesian coordinates, and $K_{I,II,III}$ is the stress intensity factor depending on the outer boundary conditions, i.e., applied load and geometry, which give the grade of stress concentration at the tip of a crack of length a , at given loading conditions (for notations see Figure 2.6).

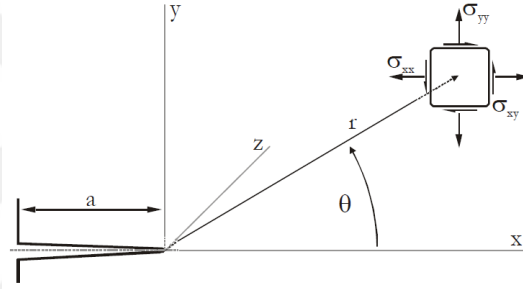


Figure 2.6 Crack tip coordinates and stress state in terms of the Cartesian system

There is a maximum limit, or threshold value, for the stress intensity factor (K), called fracture toughness (K_C). Crack initiation will occur when the stress intensity factor reaches a critical value, called fracture toughness, K_C . This value is also known as the critical stress intensity factor and is determined at the time of crack extension. As a measure of the fracture resistance property of rock to withstand crack propagation, higher fracture toughness indicates an increased fracture resistance to crack extension. The fracture toughness (K_C) of a material is also characterised by the energy per unit area required to create new crack surfaces, and thereby propagate a crack through the material.

Corresponding to the three fracture modes—mode I, opening mode; mode II, shearing mode; and mode III, tearing mode—there are three kinds of defined fracture toughness of rocks: mode-I fracture toughness (K_{IC}), mode-II fracture toughness (K_{IIC}) and mixed-mode (I–II) fracture toughness (Whittaker *et al.*, 1992).

2.5.1 Methods of mode I fracture toughness testing

Rock is a brittle material and because all brittle materials are weak in tension, generally mode I (tensile mode) is the most critical loading mode in rock mechanics applications. A number of methods have been suggested to determine mode I fracture toughness of rock materials (Chong & Kuruppu, 1984; Fowell *et al.*, 1995; Ouchterlony, 1980). The SR specimen developed by Barker (1977), Chevron Bend specimen developed by Ouchterlony (1980) and CCNBD specimen developed by Fowell *et al.* (1995) have been incorporated into a standard testing method for the fracture toughness measurement of rock by the International Society for Rock Mechanics (Baek, 1994; Ouchterlony, 1988).

The SR method was introduced by Barker (1977) and used to evaluate just mode I fracture toughness values. The specimen is a typical core with a machined notch along the rod axis. The external load is applied to the notch mouth to split the specimen into two identical halves. Figure 2.7 shows schematically the specimen configuration and the loading and dimension notations.

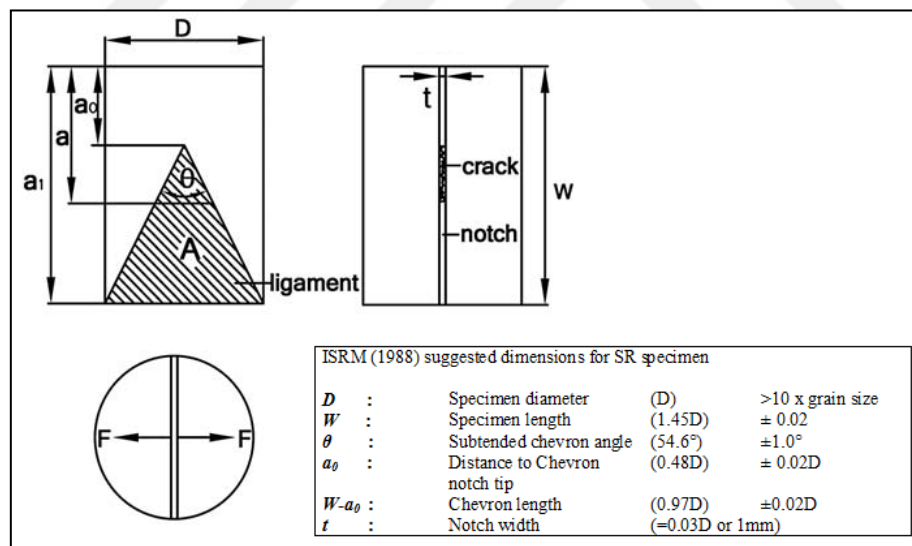


Figure 2.7 Chevron notched Short Rod specimen and associated notations

The SR test has two levels: level I and level II. Level I testing require only recording of the peak load, which is used to evaluate the level I fracture toughness. This requires that the test should be performed under load control with an average stress intensity factor rate K_{SR} of more than or equal to $0.25 \text{ MPa}\sqrt{\text{m}}/\text{s}$. Level II testing requires the recording of the complete load versus the Crack Mouth Opening Displacement (CMOD) curve. This requires that the test should be performed in

displacement control with a CMOD rate faster than $0.017 K_{SR}(E\sqrt{D})m/s$, where K_{SR} is level I fracture toughness, E is Young's Modulus and D is the diameter of the specimen.

Fracture toughness calculation is achieved by considering the following equation, which was proposed by ISRM (1988) for Level I testing:

$$K_{SR} = \frac{C_K 24.0 F_{max}}{D^{1.5}} \quad 2.14$$

where K_{SR} is the fracture toughness (MPa $\sqrt{m}$), F_{max} is the failure load (MN), D is the specimen diameter (m) and C_K is the correction factor to account for the size variation of the specimen (Ouchterlony, 1988).

The level II fracture toughness, which accounts for the effect of nonlinearity, is determined depending on whether or not the two unloading lines span the evaluation point (ISRM, 1988).

Applying direct tensile loading to a disc type specimen (with or without a notch or hole) loaded with compression through its diametral plane, which then fails in tension, has been a popular method for fracture toughness determinations. The Cracked Chevron Notched Brazilian Disc (CCNBD) method has been used more than other disc type specimen tests. A number of geometries related to the CCNBD have also been developed and used recently for fracture toughness testing of rock, but none of them have the same unique desirable combinations of features and advantages as the CCNBD geometry (Fowell *et al.*, 2006). Compared with Chevron Bend (CB) and Short Rod (SR) specimens, the CCNBD has numerous advantages, which include easier sample preparation, much higher failure load and low standard deviation between recorded failure loads, simpler testing procedure and ease of adaptability for pure mode I, mode II and mixed-mode fracture toughness testing. Moreover, one of the most important advantages of CCNBD specimen is that it does not need to have a machined precrack starting from the chevron tip. In order to obtain more accurate fracture toughness values, a sharp crack tip must be used. However, because the crack is usually a machined notch, crack tip bluntness can lead to discrepancies in calculated fracture toughness values. CCNBD specimens overcome this problem because they have a sharp chevron notch tip instead of a straight through notch. Thus, CCNBD specimens were used in this research to investigate the behaviour of crack tip damage under both static and dynamic loading, as a result of the advantages explained above.

The geometry of a CCNBD specimen is illustrated in Figure 2.8. The chevron notch causes crack propagation to start at the tip of the V alignment and proceed radially outwards in a stable fashion until the point where the fracture toughness is calculated. The specimen dimensions used in a CCNBD test are the initial chevron notched crack length (a_0), final chevron notched crack length (a_1), specimen thickness (B) and specimen radius (R).

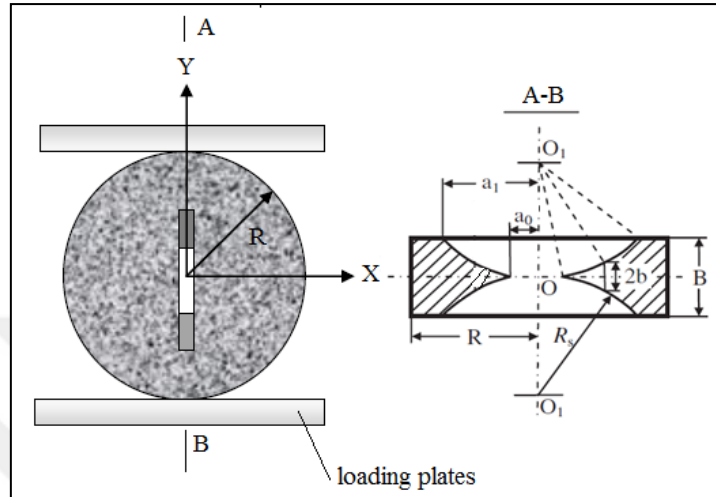


Figure 2.8 The CCNBD specimen geometry with recommended test setup

All the dimensions of the geometry should be converted into dimensionless parameters with respect to the specimen radius, R and diameter, D . The suggested standard specimen dimensions are given by the ISRM (1995). The dimensionless initial chevron notched crack length α_0 (a_0/R), dimensionless final chevron notched crack length α_1 (a_1/R) and α_B (B/R) are the three basic dimensions for the CCNBD geometrical parameters.

Other selections of specimen geometrical dimensions are possible, but in order to have a valid test, the selected dimensions for initial notched crack length (a_1) and dimensionless α_B must fall within the range outlined in Figure 2.9 (Xu & Fowell, 1993, 1994).

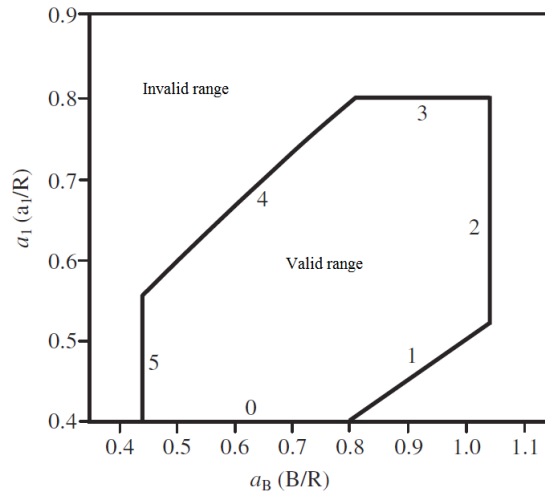


Figure 2.9 Valid geometrical ranges for fracture toughness testing (Fowell, 1994).

For fracture toughness testing, the important stress intensity factor value for the specimen is the minimum stress intensity factor, denoted as Y_{min}^* , as it corresponds to the failure load (P_{max}) recorded during testing. The fracture toughness value can then be calculated as (ISRM, 1995):

$$K_{IC} = \frac{P_{max}}{(B\sqrt{R})} Y_{min}^* \quad 2.15$$

The critical dimensionless stress intensity value Y_{min}^* , which is determined by the specimen geometry dimensions α_0 , α_1 and α_B only, can be calculated as follows (ISRM, 1995):

$$Y_{min} = ue^{(v\alpha_1)} \quad 2.16$$

where constants u and v are as listed in the ISRM suggested methods (ISRM, 1996).

2.5.2 Methods of mode II and mixed-mode I–II fracture toughness testing

For the determination of mode II fracture toughness, some experimental methods were introduced in the literature (Ingraffea, 1981; Backer, 2002; Rao, 1999; Swartz & Taha, 1991; Watkins, 1983). Very little work has been conducted on mode II fracture toughness testing, let alone any existing standard testing methods.

Mode II fracture (sliding mode) and shearing is the most common mode of failure in rock engineering. The classical mixed-mode fracture criteria are incapable of predicting Mode II fracture and at present there is no ISRM suggested method for determination of mode II rock fracture

toughness. Most of the presently available methods to measure K_{IIC} of rock materials are based on the application of a pure shear load to the specimen, regardless of the fracture pattern. For example, Huang and Wang (1985) used four-point shearing tests to obtain mode II fracture toughness of rocks. However, they found that crack propagation deviated from the notch plane and the fracture was mode I.

A new method for testing the Mode II fracture toughness under confining pressure has been developed by Backer *et al.* (2002). The new testing method is called the Punch-Through Shear Test (PTS-Test). The PTS-Test uses cylindrical samples with circular notches drilled into the end surfaces where the notches serve as friction-free initial fractures. The suggested sample geometry, and the directions of principal loading are given in Figure 2.10.

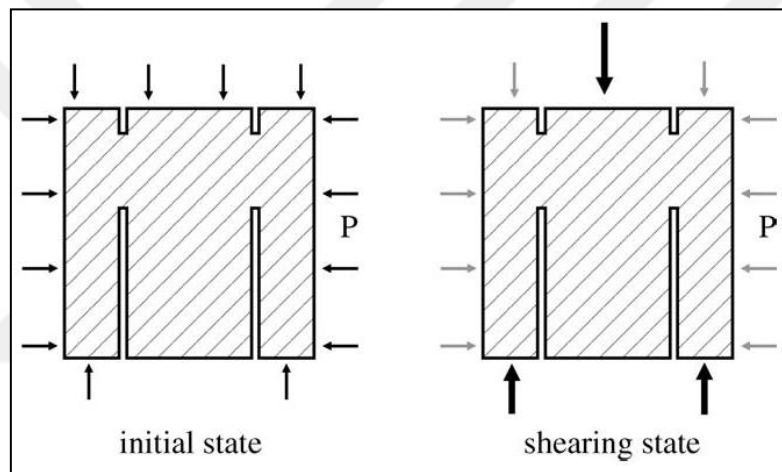


Figure 2.10 Sample geometry and principal loading setup for PTS-Test

In Figure 2.10, the left-hand side indicates the initial stress state due to confining pressure (P) (initial state) and the right-hand side indicates loading regime during application of shear stress (shearing state). The applied confining pressure acts perpendicular to the notch plane and the inner part is punched through the sample by an axial load. According to both experimental and numerical analysis results summarised by Backer *et al.* (2002), the calculated mode II fracture toughness was found to be very sensitive to the notch diameter, notch symmetry, notch length, and amount of confining pressure.

Rao *et al.* (2003) stated that the correct K_{IIC} could be determined only when a true mode II fracture occurs, no matter what kind of loading condition is applied, and should be larger than K_{IC} . They theoretically, numerically, and experimentally investigated mode II fracture of rock in order to establish a new fracture criterion for predicting mode II fracture and develop an appropriate method

for testing K_{IIC} . Figure 2.11 illustrates the state of load and stress at the crack tip for the shear fracture model. The specimen is placed between two bevelled dies, which can be set at any inclination angle, α ($0^\circ \leq \alpha \leq 90^\circ$) with respect to the horizontal direction.

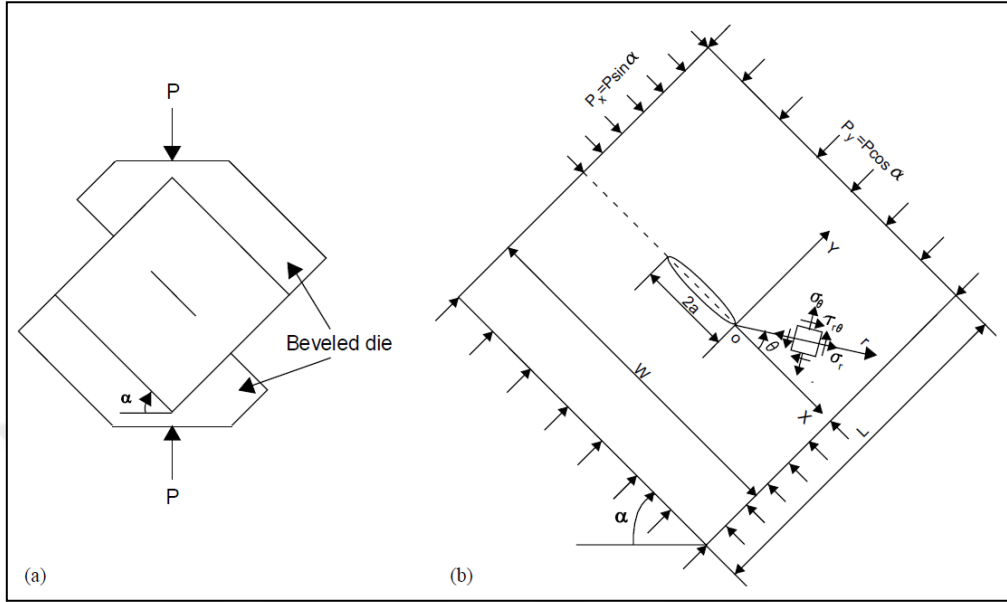


Figure 2.11 State of load (a) and stress at the crack tip (b) for the shear fracture model (Rao *et al.*, 2003)

For a double-notched specimen in shear-box, the shear stress intensity factor K_{II} can be calculated with the finite element method through the displacement approach (Whittaker, 1992) in fracture mechanics, as follows:

$$K_{II} = \frac{Q_e}{BW} \sqrt{\pi a} F\left(\frac{2a}{W}\right) \quad 2.17$$

where W and B are the specimen width and thickness, respectively. $F(2a/W)$ is the shape factor for the double-notched specimen and calculations can be found in Whittaker *et al.* (1992).

Rao *et al.* (2003) stated that the shear-box test was a potential method for determining the mode II fracture toughness K_{IIC} of a cubic rock specimen, because it could create a favourable condition for mode II fracture, i.e., $K_{II \max}$ is always 2–3 times larger than $K_{I \max}$ and reaches K_{IIC} before $K_{I \max}$ reaches K_{IC} . It also has the advantages of simple specimen geometry, fairly simple loading device, and simple calculation of the mode II fracture toughness.

In reality, rock masses include different scales of cracks and are subjected to complex loading conditions. Because of the arbitrary orientation of cracks relative to the loading direction, fracturing in brittle rocks may occur as a result of a combination of two major fracture modes; mode I and mode II. That's why the fracturing of rocks under mixed-mode loading conditions has attracted more attention in recent years (Al-Shayea, 2005; Ayetollahi & Aliha, 2007; Backer, 2002; Chang *et al.*, 2002; Rao *et al.*, 2003).

Different sample geometries have been tried to investigate the fracture toughness of rocks under a wide range of mixed-mode loading conditions, i.e., cracked Semi-Circular Bend (SCB) specimen (Ayetollahi & Alahi, 2007; Chong & Kuruppu, 1984), Arcan specimen (Hasanpour & Cohupani, 2009), shear-box test cubic specimen (Rao, 1999), etc. In general, it was found that the full range of mixed-mode loading conditions including pure mode I and pure mode II loading can be created by using an inclined crack with inclination angles (β) between 0° and 90° . Another impressive study came from Lin and Fakhimi (2009). They investigated mixed-mode fracturing in sandstone samples using a three-point bending test. They found that crack initiation did not start from the pre-existing notch crack due to the mixed-mode loading, depending on the position of the notch crack. Lin and Fakhimi (2009) observed initial damage at the tip of the notch, but the failure occurred at the centre of the smooth surface beam (without notch) in their three-point bending tests under mixed-mode loading.

The CCBBD subjected to diametral compression has been used widely to investigate mode I, mode II and mixed-mode fracture of brittle rocks (Al-Shayea, 2005; Atkinson *et al.*, 1982; Avaji & Sato, 1978; ISRM, 1995; Shetty, 1985). More commonly, if a CCBBD specimen is loaded with the crack oriented at an appropriate angle with respect to the loading direction, a mixed-mode I–II stress field can be achieved at the crack tip (Figure 2.12).

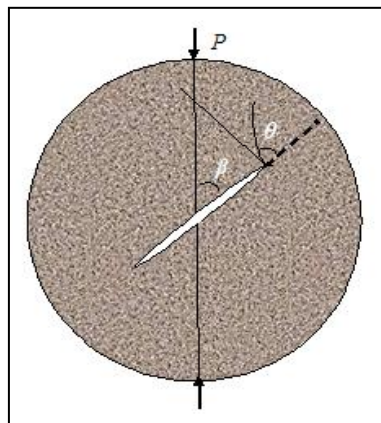


Figure 2.12 CCBBD sample under diametral compression

The related fracturing stress for mode I, mode II and mixed-mode I–II loading conditions under compression can be found, similar to the case of tension, by using the three fracture criteria. However, some researchers have provided a closed-form solution for stress distribution in cracked and uncracked disc specimens under diametral compression (Atkinson *et al.*, 1982; Avaji & Sato, 1978; Shetty, 1985). Shetty (1985) first used a CCNBD specimen to calculate both mode I and mode II fracture toughness of ceramics and developed stress intensity factor solutions for Straight-Through Brazilian Disc (STBD) specimens with a through notch. However, Atkinson *et al.* (1982) and Avaji and Sato (1978) developed dimensionless mode I stress intensity factor (NI) and dimensionless mode II stress intensity factor (NII) solutions, depending on the dimensionless notch length α (a/R) and the notch inclination angle (β) with respect to the loading direction, as the first and only solution in the literature. Atkinson *et al.* (1982) developed NI and NII solutions for the Chevron Straight Through Brazilian Disc (CSTBD) specimen given by a five-term approximation for $0.1 \leq \alpha \leq 0.6$:

$$N_I = \sum_{i=1}^n T_i(\alpha)^{2i-2} A_i(\theta) \quad 2.18$$

$$N_{II} = 2\sin 2\theta \sum_{i=1}^n S_i(\alpha)^{2i-2} B_i(\theta)$$

where T_i and S_i are numerical factors and $A_i(\theta)$ and $B_i(\theta)$ are angular constants; their calculation details can be found in the work of Atkinson *et al.* (1982). Rock fracture toughness under mixed-mode conditions, however, cannot be calculated by the ISRM suggested methods. Therefore, the STCA method, which uses SCBDC specimen geometry, was used to obtain fracture toughness under mode II and mixed-mode conditions in this study.

2.6 Cyclic Loading and Rock Fatigue

Some rock structures such as bridge abutments, dam and road foundations, and tunnel walls undergo both static and cyclic loading caused by vehicle-induced vibrations, drilling and blasting, traffic, etc. This type of loading often causes rock to fail at a lower stress. The design of such structures requires understanding of and research on rock mechanical properties under various loading conditions.

It is a known fact that cyclic loading often causes brittle materials such as ceramics and rocks to fail at a stress level lower than their determined strength under monotonic conditions (Attawel & Farmer, 1973; Burdin, 1963; Evan & Linzer, 1976). This phenomenon is commonly termed 'fatigue'. Faults, joints, bedding planes, tunnel walls, excavation roofs and ribs, bridge abutments, dam and road foundations are only a few of the natural and manmade rock structures that can be weakened by repetitive loading. Better understanding of cyclic fatigue could assist planners in preparing a more rational design that will eliminate premature failures and rock burst. Knowledge of fatigue characteristics could also help improve rock breaking methods.

Specific terms and definitions have been used, such as fatigue stress, fatigue strength, maximum applied stress (peak stress level), minimum stress level (valley value), etc., in fatigue literature, as follows (Singh, 1989):

- *Fatigue stress*: The maximum applied stress at which a rock can stand an infinite number of cycles without destruction.
- *Fatigue strength*: The stress level at which the fatigue life is 'N' number of cycles.
- *Maximum applied stress* ($S_{\max}$): The maximum stress experienced by a sample during a cyclic load test. It is normally defined as a percentage of normally computed mean uniaxial compressive strength of the rock (Figure 2.13).
- *Minimum stress level* ($S_{\min}$): The minimum stress value in a cyclic loading. Its value is always higher than zero (Figure 2.13).
- *Stress amplitude*: The arithmetic mean of the maximum and minimum applied stress levels of a cycle. The mean stress level of a cycle is defined as the algebraic difference between maximum and minimum stress levels (Figure 2.13).

The total number of cycles applied to each specimen is known as 'number of cycles' (N) and the total number of cycles per second is defined as 'cyclic frequency'.

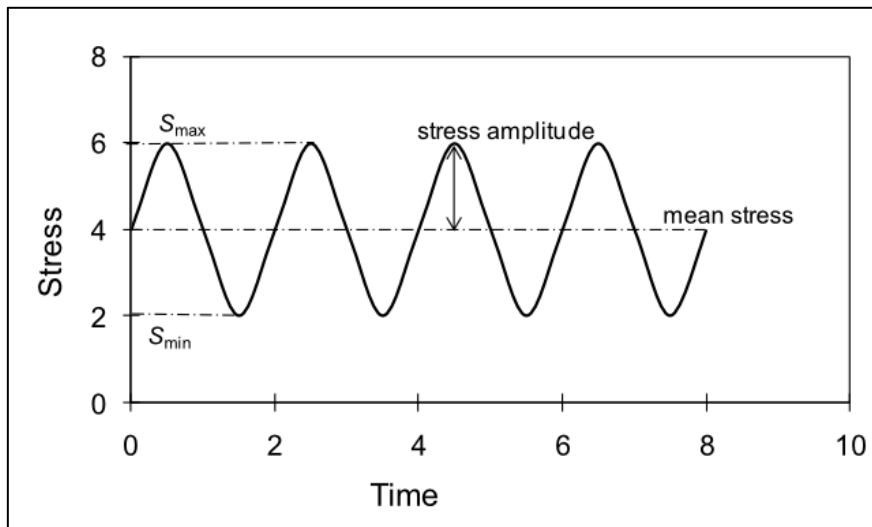


Figure 2.13 Definitions of cyclic loading

The relation between these parameters, known as the S–N curve in fatigue literature, is that the number of cycles to failure (N) or its logarithm is inversely proportional to the peak stress (S). During fatigue tests, the machine is kept running within a particular stress range, alternating the stress until the test piece fails. The number of cycles of stressing up to failure is recorded and the test is repeated for different stress ranges. There is a specific stress amplitude called the fatigue limit, for which the material will endure an infinite number of stress cycles. For any stress amplitude greater than this endurance limit, failure will occur if the material undergoes a sufficient number of stress cycles (Figure 2.14).

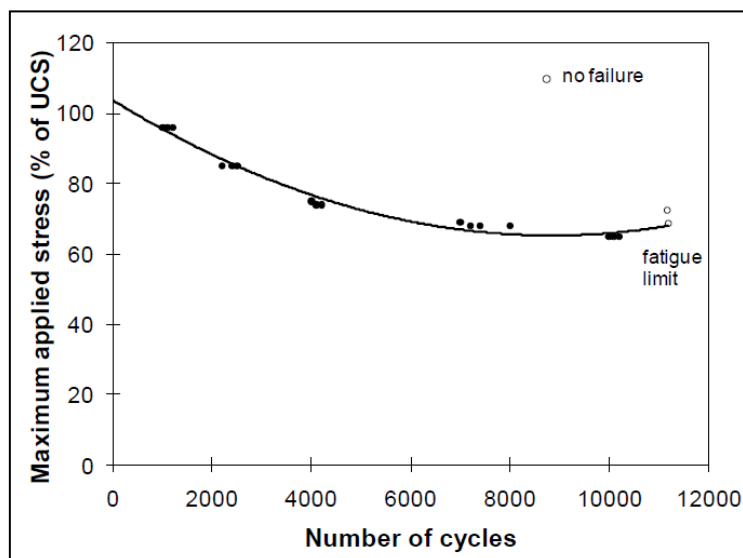


Figure 2.14 Typical S–N graph for Indiana limestone (Costin & Holcom, 1981)

Mechanical behaviour of rock under static loading has been thoroughly investigated. However, rock reaction to cyclic, repetitive stresses resulting from dynamic loads has been generally neglected,

with the exception of a few rather limited studies (Attawel & Farmer, 1973; Bagde & Pedros, 2005; Gattelier *et al.*, 2002; Haimson, 1978; Tao & Mo, 1990). The effect of the fatigue mechanism on rock failure is still the subject of much research in fracture mechanics (Lavrov, 2001; Zang *et al.*, 2008). Research has indicated that, under static loads, the largest or critical crack is responsible for the failure, while under dynamic loads, a single crack is not sufficient to result in failure of the rock. A typical feature of fatigue in experimental tests is that the repetition of loading cycles produces a progressive accumulation of permanent strain in the specimen.

A limited amount of research on rock fatigue was published before 1970. Among the first studies of cyclic fatigue of rock was Burdine's (1963) investigation to determine the fatigue strength of Berea sandstone under uniaxial and triaxial compression. He found that the monotonic compressive strength could be weakened by as much as 24% due to cyclic loading. Hardy and Chugh (1970) reported on studies to determine the fatigue life and fatigue strength of Barre Granite, Tennessee Sandstone, and Indiana Limestone samples. According to their results the fatigue strength of the tested rocks varied between 65% and 80% of static uniaxial compressive strength. Important rock fatigue research was carried out by Attawel and Farmer (1973) on concrete, mortars and on specimens under cyclic compression loading. They reported that the fatigue strength appeared to be reduced by 50 to 70% of the uniaxial compressive static strength.

Haimson and Kim (1971) performed experiments on two types of marbles; White Tennessee Marble and Georgia Marble. Their results showed that the fatigue strength of these materials was around 75 to 80% of the static compressive strength. However, they found no apparent influence on the results as a result of the range of frequencies used. A different approach to rock fatigue was made by Singh (1989) who obtained strain hardening with greywacke samples from the Australian Flagstaff Formation having a mean Uniaxial Compressive Strength (UCS) of 185 MPa. He found a strength decrease up to 17%, however, he observed strain hardening up to 29% by using cyclic loading values lower than the obtained fatigue limit of the samples. Moreover, he found that the fatigue life of the rock increased with decreasing stress amplitude, and the number of cycles to failure increased on a logarithmic scale.

Rock fatigue research also has extended into the rock cutting area. An early attempt to assess the effect of cyclic loading on rocks due to the action of mechanical cutters was made by Roxborough (1968, 1973). Research into the effect of vibratory cutting using static and percussive tools was done on coal by Roxborough (1968). He found that cutting force can be significantly reduced by the use of vibration, with frequency and cutting speed being the major variables.

Gottlieb and Moore (1981) looked for means to improve efficiency for the vibratory cutting of brown coal, following an earlier study by Roxborough (1968). They found that the effect of vibration in the cutting tooth reduced the magnitude of the cutting forces between 15% and 23%. In the last decade, Hood and Alehossein (2003) described a novel method of rock cutting; Oscillating Disc Cutting (ODC). ODC uses a free-rolling disc cutter, which undercuts the rock. This failure mode requires stresses that are an order of magnitude lower than for compression. Hood and Alehossein found that the force used to cut hard rock was reduced up to 70 to 80% relative to the usual techniques.

Bagde and Petros (2009) investigated the fatigue and dynamic energy behaviour of sandstone samples. They concluded that dynamic strength was related to the features of dynamic loading, and the dynamic strength had a loading rate effect. Their research determined that the average Young's modulus decreased with increasing loading frequency and amplitude. They also concluded that the dynamic axial stiffness of the rock decreased with increasing frequency.

Amplitude and frequency of each cycle are the main operational parameters in fatigue research and they are very effective parameters of strength reduction in rocks due to fatigue. Attawel and Farmer (1973) performed cyclic loading tests on dolomite specimens with frequencies between 0.3 Hz and 20 Hz. They revealed that the crack propagation process was found to be very strain dependent and that the rock would more readily succumb to low frequency dynamic stresses of higher amplitude (but within the stable crack propagation regime) than to high frequency dynamic stresses of low amplitude.

Ishuaka *et al.* (1990) reported that, although fatigue failure is natural for rocks, the reduction in strength of different rocks due to repeated loading differs greatly. They performed two different sets of cyclic tests: (1) fatigue failure dependence on frequency under cyclic loading of identical stress, and (2) frequency dependence of fatigue failure under equal stress ratio cyclic loading. They found after the first series of tests that the fatigue life increased with increasing frequency. In contrast, they found in their second series of tests that the fatigue life (number of cycles to failure) was constant under any excitation frequency.

Xu (2002) carried out systematic experimental studies on red sandstone and Da-ye Marble under cyclic loading with a 0.06 to 3 Hz frequency range, in the pre- and postpeak loading regions. He concluded that there were different macrofracture properties for specimens tested at low and high

cycle frequencies. At high frequency, the fracture of specimens subjected to cyclic loading postpeak was more serious than that found prepeak. The number of cycles to failure decreased with increase of the cyclic load level and decrease of loading frequency because of the natural structural properties of the rock.

The generally accepted mechanism for time-dependent weakening of rocks currently is referred to as 'stress corrosion'. Stress corrosion crack growth occurs because the chemical action of an environmental agent, such as water, or other weakening mechanical action, such as fatigue, weakens the strained bonds at crack tips and so facilitates crack propagation under lower stress levels. Costin and Holcomb (1981) indicated that decreasing the amplitude means increasing the mean stress during each cycle, which might increase the amount of stress corrosion and decrease the amount of damage produced by cycling. Stress corrosion is a time-dependent mechanism that is most sensitive to the mean stress level, whereas cyclic fatigue is most sensitive to the amplitude of the stress cycles. According to Costin and Holcomb (1981), if the maximum stress was lowered until stress corrosion becomes insignificant over the duration of the experiment, then damage could be controlled by varying the cycle amplitude. At high mean stress and low cycle amplitude, stress corrosion dominates; whereas at low mean stress and high cycle amplitude, cyclic fatigue dominates (Costin & Holcomb, 1981).

However, this does not lead to the conclusion that a decrease of amplitude will result in faster failure of the specimen. In fact, if the amplitude is decreased, the size of the hysteresis loop would be narrow; hence the damage produced by cycling would be strongly reduced. If the amplitude is increased, the size of the hysteresis loop would increase rapidly, which would result in strong damage produced by cracking (Tao, 1990). Also, the larger the amplitude, the faster lateral deformation of the specimen develops (Tao, 1990). One way to reduce the damage in each cycle is to reduce the maximum stress. For large amplitude cycles, lowering the maximum stress is doubly effective at reducing damage because both the stress corrosion and the fatigue effects are reduced (Costin & Holcomb, 1981). For maximum stresses greater than the threshold, lowering the maximum stress greatly decreases the damage rate (Holcomb, 1979).

Petros and Bagde (2003) carried out an investigation to assess the strength and deformation characteristics of sandstone samples under cyclic loading. They concluded that changes in the modulus of deformation with frequency were very small, whereas the dependence of the modulus of deformation on the amplitude of deformation, at a constant frequency of dynamic loading, was very strong.

The results of previous studies, in terms of the strength reduction of rock specimens subjected to cyclic loading, revealed by the conventional S—N approach, have demonstrated that brittle rocks can be strongly affected by cyclic loading. However, most of the rock fatigue research has been focused on uniaxial compressive strength degradation under cyclic loads. Information regarding the dynamic tensile properties of rocks is of considerable importance in assessing the stability of rock structures under dynamic loads. It is also important in determining rock breakage and fragmentation under explosive and percussive excavation. There is very limited research on the response of tensile strength of rocks to cyclic loading (as opposed to dynamic loading, such as explosive loads and impact loading). For quasi-brittle materials such as concrete and rock, cracking is the major cause of material failure in many cases. The most fundamental parameter in fracture mechanics is fracture toughness, which describes the resistance of a material to crack propagation. Fracture toughness is an important material property that corresponds to the critical state of the stress intensity factor required for crack initiation and subsequent propagation. As a result, assessment of resistance to crack propagation would be crucial in understanding the behaviour of structures involving brittle materials.

Based on LEFM, two main mechanisms were proposed to explain how rock fails under cyclic loading conditions: (1) fracture toughness decreases with time (stress corrosion); and failure occurs when it reaches the maximum value for the stress intensity factor imposed by the load (Atkinson, 1984; Dauskardt *et al.*, 1990). (2) the maximum value for the stress intensity factor increases even though the maximum external load is kept constant. Failure takes place when it reaches the fracture toughness (Dauskardt *et al.*, 1990).

The physical processes occurring at the crack tip are complex and probably stress corrosion and increasing the stress intensity factor at the crack tip under a constant load occur (Celestino *et al.*, 1995). Celestino *et al.* (1995) performed diametral compressive cyclic loading tests on Brazilian granite disc samples to investigate mechanisms of rock fatigue and creep in rocks. They concluded that rock fracture toughness determined during quasi-static load was consistent with fatigue and creep. Therefore, no significant decrease of fracture toughness before failure under any load condition was observed (Celestino *et al.*, 1995). However, possible crack propagation under $K < K_C$ conditions, which contradicts the theory for crack propagation under static loads proposed by LEFM, was observed, and the relationship between crack velocity and stress intensity factor under creep was obtained.

Creep failure of brittle rock is usually attributed to stress corrosion. Although usually associated with static (creep) load, stress corrosion may also take place during cyclic load, with lower intensity due to the shorter elapsed time of the applied load (Celestino *et al.* 1995; Scholz & Koczynski, 1979). However, some experimental results for brittle materials, e.g., ceramics, showed that the crack propagation rate under cyclic tests could be predicted by integrating the contribution of stress corrosion, with no influence of fatigue (Evans & Fuller, 1974).

The fracture evolution process (initiation, propagation and coalescence) in cracked rock was observed and the force–displacement curves of cracked rock were measured under uniaxial cyclic loading by Zang *et al.* (2008). Their experimental results indicated that the fatigue deformation limit corresponding to the maximal cyclic load was equal to that of the postpeak locus of the static complete force–displacement curve. The fatigue deformation process can be divided into three stages: initial deformation, constant deformation rate and accelerative deformation; the time of fracture initiation, propagation and coalescence corresponds to the change to irreversible deformation.

2.6.1 Fatigue damage mechanism

A typical feature of fatigue in experimental tests is that the repetition of loading cycles produces a progressive accumulation of permanent strain in the specimen, rather than any significant decay in the material's elastic modulus. In most research, acoustic emission and specimen photomicrography suggest microfracturing as the principal mechanism in the fatigue failure of rock, with distinct differences between cyclic compression and cyclic tension (Haimson, 1978). According to Haimson's (1978) research, fabric changes in uniaxial compression appeared to be dominated by grain boundary loosening and intergranular cracking in the first few cycles, followed by a stage in which no additional damage seemed to occur, although some crack extension probably did take place. Crack coalescence, widening and faulting resulting in fatigue failure were eventually observed.

According to research by Zoback and Byerlee (1975), dilatancy can be an apparently stable process, and the onset of dilatancy is not affected by repeated cycling under triaxial conditions. Their results showed that a sample was significantly axially compressed and radially expanded (backsliding and crack closure had not completely occurred) to remain stable after the first cycle. However, when hydrostatic pressure was removed, the normal stress acting across grain boundaries was reduced and backsliding and crack closure progressively took place.

In general, for a prescribed stress higher than a certain threshold in cyclic loading tests, damage increases rapidly in the first few cycles, then stabilises and increases again in the last few cycles. In contrast, for a prescribed axial stress lower than this threshold, damage stabilises without additional irreversible dilatant volumetric strain. Then the response of the rock becomes independent of the cycles (Costin & Holcomb, 1981). This behaviour has been encountered in a wide variety of brittle rocks. If the maximum applied stress level on cyclic loading does not cause rock failure up to an infinite number of cycles, the rock becomes 'strain-hardened', i.e., its UCS increases. Thus, if the maximum applied stress is below the fatigue stress level, cyclic loading will always produce strain hardening but not fatigue failure (Singh, 1989).

Eberdhard and Stead (1999) carried out damage-controlled cyclic loading tests to examine the accumulation of damage and its influence on the deformation and fracture characteristics of granite samples by using two type of tests, according to the 'crack damage threshold' value: (1) damage control cyclic loading testing above the crack damage threshold, and (2) damage control cyclic loading testing below the crack damage threshold. Results from the type 1 test above the crack damage threshold showed possible mechanisms by which the coalescence of interacting cracks would be accompanied by large plastic strains as the bridging material between the cracks weakens and collapses (Figure 2.15). The coalescence of smaller cracks would result in the development of new, effectively longer cracks in which the crack tip would be located in less damaged and therefore stiffer material. Increased load would be required during the next damage increment to develop the process zone around the tips of these newly formed cracks. This process may repeat itself during subsequent load–unload cycles (Eberdhard *et al.*, 1999).

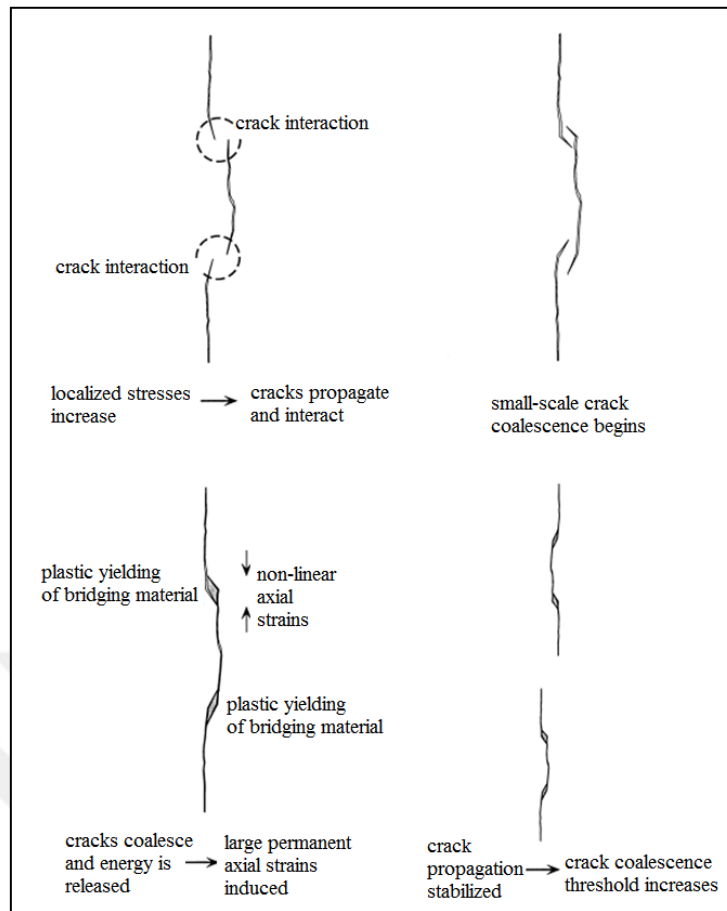


Figure 2.15 Conceptual model of crack coalescence accompanied by large permanent axial strains (After Eberdhard *et al.*, 1999)

Results from the second damage controlled test, in which cyclic loads were kept below the crack damage threshold, revealed that the slow development of the microcrack population resulted in the initiation and propagation of new cracks with each damage increment. Failure occurred when the crack population reached a state, both in density and size, in which large-scale crack interaction, coalescence, and unstable propagation ensued (Eberdhard *et al.*, 1999).

Evans (1980) illustrated that crack tip plasticity was a necessary but not sufficient condition for the onset of fatigue crack propagation in fine-grained ceramics, when the principal crack length substantially exceeds the particle diameter. However, generalised crack tip plasticity does not occur in most ceramics. Analogous fatigue treatments for ceramics must therefore invoke alternate modes of plasticity; such as indentations at crack surface asperities (Figure 2.16). Alternatively, a completely different source of fatigue must be sought (Evans, 1980). As shown in Figure 2.16, plasticity could develop whenever a macrocrack is subjected to normal compression, termed the macrocrack fatigue mechanism. Contacting asperities form on the crack surfaces because of relative displacements that occur when a crack relieves the localised residual stresses created by thermal

contraction anisotropy in ceramics. Very large stresses develop at the asperity contacts under an applied compression; which could exceed the hardness of the material. The resultant plasticity may lead to the formation of lateral cracks during unloading. These cracks could be of sufficient length to constitute an extension of the crack front.

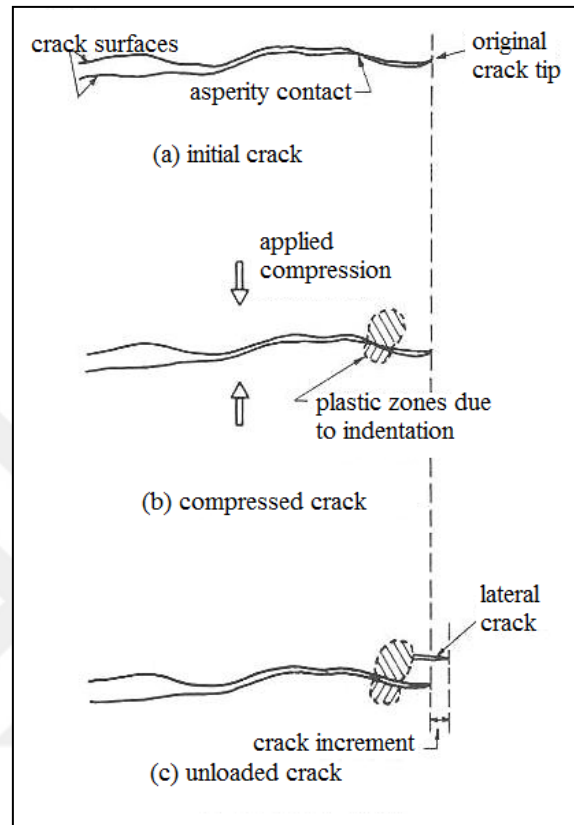


Figure 2.16 Formation of lateral cracks at crack surface asperities

Dal Pino *et al.* (1999) studied the fatigue damage of marble using SEM analyses. They obtained pictures from partially loaded–unloaded samples and undamaged samples under constant cyclic loading. After the SEM analyses, they observed a general softening of the rock cohesion, which appeared to be due essentially to the detachment of the calcite grains from one another, rather than to crystal fracture along cleavage planes. They reached two main conclusions when comparing fatigue failure with quasi-static rupture. First, the number of pieces produced was much greater in fatigue than in monotonic loading and, second, the fracture surfaces in fatigue had a crumbling aspect and could be reduced to powder by finger pressure (Figure 2.17), whereas in monotonic loading they were sparkling, as typically results when rupture of the crystals occur along cleavage planes (Figure 2.18).

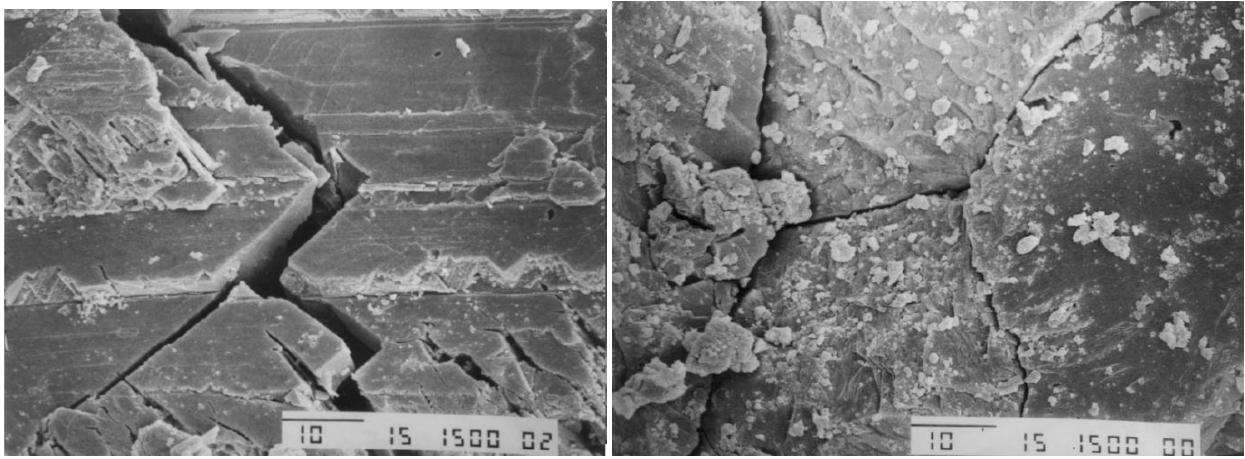


Figure 2.17 Intergranular and intragranular fractures in a cyclically broken marble specimen

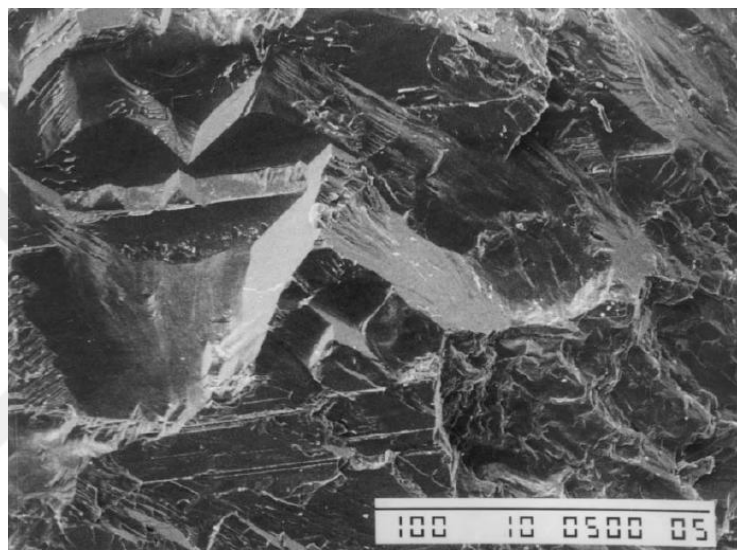


Figure 2.18 Crystal cleavage fracture in a monotonically broken marble specimen

Carfagni and Salvatore (2000) carried out research on three different qualities of marble samples. The typical feature of their experiments was that repetition of low cyclic loading produced a progressive accumulation of permanent strain in the specimens, rather than any significant decay in the material's elastic modulus. Their research included scanning electron microscope studies, which revealed that there were two different phases in the marble's behaviour. The first was characterised by calcite grain decohesion, i.e., the grains remained integral, but detached along their interfaces upon repeated load cycles (grain decohesion). In the second phase, there were evident signs of transgranular microcracking. These two distinct phases were shown to correlate with different stages in the material's macroscopic response to cyclic loading.

Gattelier *et al.*, (2002) investigated mechanical damage of an anisotropic porous rock in cyclic triaxial tests. They found that cyclic tests displayed two inelastic mechanisms in the sandstone, namely compaction and microcracking, with distinct mechanical effects. Compaction, which was a

result of the collapse of pores and readjustment of grains into a denser packing pattern in the direction of axial stress, was the reason for axial modulus increase and occurrence of contracting irreversible volumetric strain. In contrast, dilatancy, which was due to intergranular cracks and subsequent movement of grains, produced a decrease in the transverse moduli and the appearance of dilatant, irreversible volumetric strain.

Cardani and Meda (2004) analysed the tensile behaviour of marble at failure under monotonic and cyclic loading by using three- and four-point bending tests. They confirmed the decohesion of the calcite grains with SEM analysis and showed fatigue phenomena (cyclic striation) in the compression zone, due to cyclic loads.

2.7 Chapter Summary

The existing theories and models proposed for deformation and failure of brittle materials under indirect tensile monotonic loading can be attributed to the development of stress-induced microfractures under indirect tensile cyclic loading. The practical application of these theories (i.e., Griffith's crack theory, stress corrosion, etc.) primarily involves the determination of stress thresholds at which a crack will initiate and propagate. Thus, concentration on the area at the tip of a crack has allowed for the correlation of the microfracturing process with the progressive failure of a rock material under cyclic loading. Tensile stress-related microfracturing can be equated with irreversible damage and applied in a model that quantifies the tensile strength degradation and fracture toughness of brittle rock material.

CHAPTER 3

EQUIPMENT AND MATERIALS

Most of the tests in the current research were carried out on Brisbane tuff, because it is a host rock of Brisbane's first motorway tunnel, CLEM7, from which core samples were obtained. Limited tests were carried out on sandstone and granite specimens to extend the research to different rock types. In order to determine mineral content, texture and cement of Brisbane tuff, petrographic analyses were done. Moreover, uniaxial compressive strength, Brazilian tests and Shore Scleroscope hardness tests were conducted on Brisbane tuff specimens to determine the mechanical characterisation of Brisbane tuff.

Because the Instron extensometer is not very sensitive for dynamic strain reading, special Data Acquisition (DAQ) software called Extensometer Data Acquisition Program (EDAP) was designed for cyclic tests performed on Brazilian disc specimens.

To examine the major characteristics of the damage process at the microscopic level, SEM imaging was used. Tips of the chevron notch crack in CCNBD samples tested under both static and cyclic tests were examined with SEM. Tested samples and failure surfaces were examined directly with SEM without using prepared thin-section samples or sample coating because creation of extra new microcracks is possible during thin-section sample preparation or high vacuum SEM imaging.

3.1 Loading and Deformation Measurement Equipment

For compressive loading tests, two different model Instrons were used with 1000 kN and 50 kN load cells that can be configured separately according to the test type, i.e., static loading or cyclic loading. For cyclic testing, both Instron types were equipped with FastTrack 8800 Materials Test Control System. The FastTrack 8800 controller is available in both a single axis and a multiple axis configuration. The FastTrack Console forms the user interface between the user and the materials test control system. It provides all the communications functions between the user's test controller and application programs. Also, the FastTrack Console has multitest capabilities when used as an interface. During testing, FastTrack Console controls the test toolbar that is used to display a summary view of any tests that are being carried out. The test toolbar is a dedicated area of the screen that cannot be minimised, closed or overlaid by any other window. Included with the

FastTrack Console software is the DAX data acquisition software. When used in combination with FastTrack Console, DAX software can be used to gather information about a test as it progresses.

For static loading tests, Instron Bluehill 2 Software was used to control the Instron for both compression and tension tests. Bluehill 2 is a simple-to-use, tabbed user interface. It consolidates testing, test method definition, test reports and system administration, making the entire testing process very easy to manage.

Diametral axial displacement parallel to the loading direction in disc specimens was measured by means of an extensometer with 50 mm gauge length installed in the Instron loading frames. The crack displacement was measured as CMOD across the crack mouth. A clip gauge for measuring the notch opening was attached to the knives. An Instron 2670 series crack opening displacement gauge was used to measure CMOD. The gauge length was 10 mm and maximum travel was 2 mm. The gauge meet the requirements set out in British standard BS 5447 and American standard ASTM 399 70T. In addition, cylindrical UCS test specimens were wired with two lateral and two axial CEA-06-500UW-350 model Micro Measurements Division (USA) strain gauges per specimen, for measuring both axial (longitudinal) and circumferential strain to determine Young's modulus and Poisson's ratio.

Because the Instron extensometer is not very sensitive for efficient dynamic strain reading during dynamic cyclic loading, special DAQ software called EDAP was designed for cyclic tests on Brazilian disc samples. The program acquires four channels of data: load, displacement and axial/lateral strain readings. The Brazilian disc sample has lateral and axial strain gauges attached to it on faces perpendicular to each other, lateral and axial to the force applied to the sample. These two strain gauges are two of the data channels. The other two channels are direct voltage outputs from the Instron extensometer, which are indicative of the load applied and the position of the Instron probes. The designed program records these four channels in a comma-spaced variable (.csv) spreadsheet file alongside a time-stamp for each sample. The user is able to specify the sampling rate of the program and is able to control when data acquisition commences and terminates.

The hardware modules used to capture the data were purchased by the Civil Engineering department of the University of Queensland from a company called National Instruments (NI). The two modular cards are both C-series modules and they slot into a C-series chassis. The first card is an analogue voltage data acquisition card (NI 9205) and the second card is a strain gauge data

acquisition card (NI 9236) used for measuring 350 Ω strain gauges in quarter-bridge configuration (Figure 3.1). The C-series chassis is powered independently from the Instron extensometer and controlling computer and connects to a laptop by means of a USB cable.

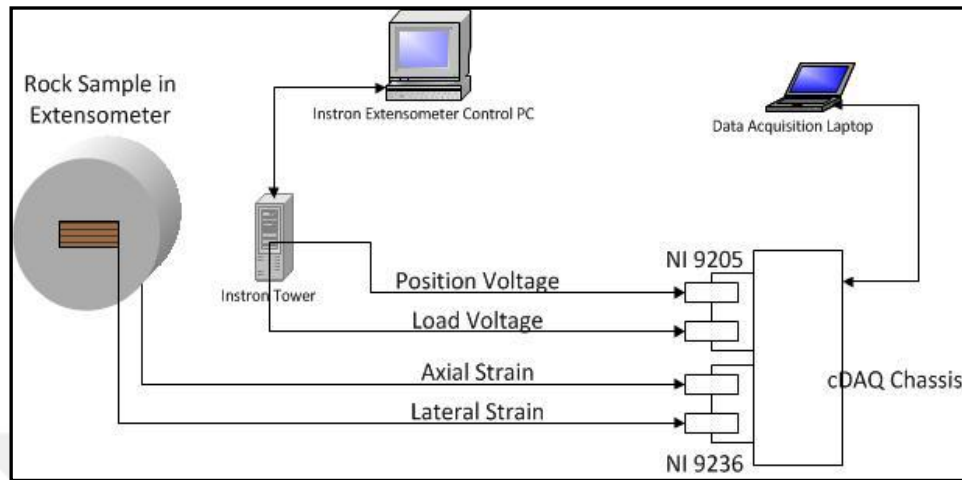


Figure 3.1 DAQ system

The Extensometer Data Acquisition program has been specifically designed to acquire data during cyclic load testing of rock samples. The Instron control software only has the ability to run a cyclic test for two hundred seconds, and then the machine will need to reset before loading the next stage of the cyclic test. This resetting produces null points in the cyclic test and the data acquisition program has the ability to remove these null points from the final data set. Fortunately, the program also produces a raw data file with the null points included, so it may be possible to change some of the test parameters of the program to remove the null points at a later stage. During the development of the software, every care was taken to make the program as simple as possible to use (Figure 3.2). The user will be presented with the front panel of the program when the program starts. An 'OFFSET' function has been provided to the user to null the initial offset voltage of the Wheatstone bridge and allow the user to read accurate strain values when running the experiment (Figure 3.2).



Figure 3.2 Three reading channels for load, axial/lateral strain and position

3.1.1 Cyclic loading tests

Single Axis MAX (SAX) is a software application that is part of the FastTrack 8800 Materials Test Control System, designed to allow for performing single-axis cyclic tests on specific testing systems. It is possible to use MAX to perform a variety of tests, from simple fatigue tests to complex, pseudorandom simulations. MAX is easy to use and has been designed to run with other applications.

Two types of cyclic loading tests were performed in this research: (1) sinusoidal cyclic loading with constant amplitude and mean stress level (Figure 3.3) and (2) increasing mean level cyclic loading with constant amplitude (Figure 3.4). Both types of cyclic tests were performed with 1 Hz loading frequency.

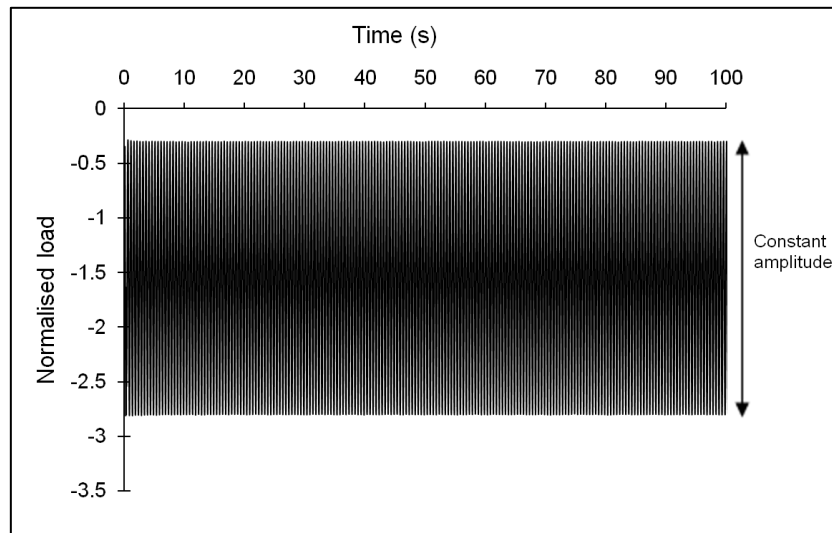


Figure 3.3 Sinusoidal cyclic loading—constant mean stress with constant amplitude

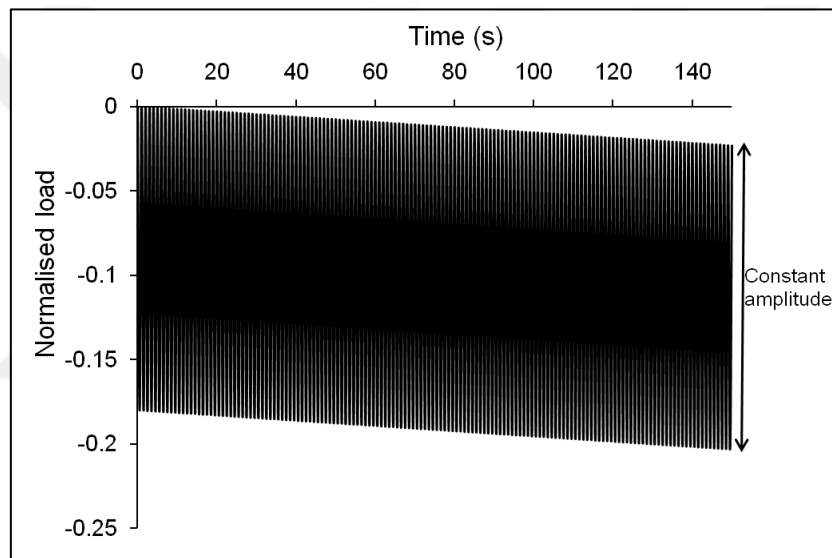


Figure 3.4 Increasing cyclic loading—increasing mean stress with constant amplitude

Sinusoidal cyclic loading tests were performed easily by using a sinusoidal waveform generator ; however, performing the dynamic cyclic loading tests with increasing mean stress level was not easy because they used a random user defined waveform generator. The SAX software allows one to use and run an existing test or design a waveform by importing user defined waveform data via RDF files for a random user defined waveform generator. The software uses the following file extensions:

- .csv Data logging output from MAX in CSV format
- .dat Cycle count and peak value tracking information, output in CSV format
- .rdf Sampled data input file created by importing a CSV file; contains filtering information

- .seq Sequence file listing test files to be run
- .tst Test file contains all the test information

These file formats were used to design test sequences to perform cyclic loading with increasing mean stress level. SAX software has four types of waveform generator; sinusoidal, triangular, square and random definition. Generating the random definition waveform requires a special configuration to use the waveform generator efficiently for designing custom waveforms. The software allows you to import data to generate complex demand waveforms in two ways:

1. Turning Point—allows you to define a cycle with up to 400 turning points that are linked with ramps or haversines; and
2. Sampled Data—up to 3200 points of field data (such as the shock of a suspension arm negotiating a bump) may be repeated as required.

New waveforms may be created or the sampled data modified using a spreadsheet package such as Microsoft Excel, and the resultant file saved in CSV format. This data can be imported into the software and converted to an RDF file. In contrast, preparing each test sequence was limiting in either of those two options. For example; 20 individual test sequences need to be prepared to run a cyclic test with 2000 cycles at 1 Hz loading frequency. After preparation of each test sequence with Microsoft Excel, each test sequence file must be imported as a .csv file into the waveform generator. The imported .csv files were saved as .RDF files to design the prepared test sequences. The .RDF test sequence files were then designed in the SAX program by adding the sequences from the first to the final sequence file. Then, the designed sequences were saved as a single input test data file with .tst file extension to perform the cyclic tests up to failure. All these data input processes were quite time consuming.

3.2 Sample Preparation

A key element in rock testing is accurate sample preparation. The sample preparation and testing procedure conformed to requirements of the International Society for Rock Mechanics (ISRM, 1981). Laboratory work included a number of tests: UCS, Brazilian Indirect Tensile Strength (BTS) tests, Direct Tensile Strength (DTS) tests, fracture toughness tests and Shore scleroscope hardness tests.

According to ISRM standards, at least five replicates should be tested to reliably determine UCS, Young's modulus and Poisson's ratio; indirect tensile strength and fracture toughness. For determination of USC, Young's modulus and Poisson's ratio, sample preparation involved coring and trimming to form cylindrical test specimens conforming to ISRM recommendations (that is, cylindrical specimens 52 mm in diameter by 134 mm high to give a height to diameter ratio of 2.5 to 3.0, a ratio of specimen diameter to that of the largest rock grain $> 10:1$ and ends of the specimen that are flat to within 0.02 mm). Both bottom and top ends of USC samples were ground to supply the ISRM requirements to eliminate end effects on the test (Figure 3.5).



Figure 3.5 Grinding the ends of the UCS samples

Specimens were wired with two lateral and two axial CEA-06-500UW-350 model Micro Measurements Division (USA) strain gauges per specimen, for measuring both axial (longitudinal) and circumferential strain to determine Young's modulus and Poisson's ratio (Figure 3.6). For indirect Brazilian tensile tests, sample preparation involved coring and trimming to form discs conforming to ISRM recommendations (that is, discs 52 mm in diameter by 26 mm thick, a diameter to thickness ratio of 2.0).

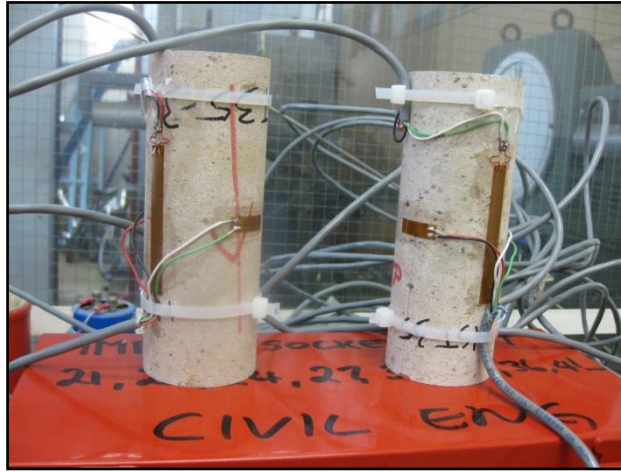


Figure 3.6 Two successfully strain-gauged UCS samples

According to ISRM suggestions, the required minimum diameter of the disc for CCNBD specimens was found to range from 20 to 30 mm for all selected rock types. The samples were already cored in 52 mm diameter, hence; it was decided to adopt the CCNBD method's requirements to our samples according to the valid range of ISRM (ISRM, 1995) for all specifications for the preparation of samples. For proper specimen preparation in the CCNBD method, all parameters of the geometry should be converted to dimensionless forms with respect to the radius and the diameter of the specimen. The two most important selected dimensions, dimensionless final notched crack length (α_1) and dimensionless quantity, α_B , must fall within the range outlined in Figure 3.7 (ISRM, 1995). All specimen geometries used in this study were in the validity range shown in Figure 3.7 as a red circle.

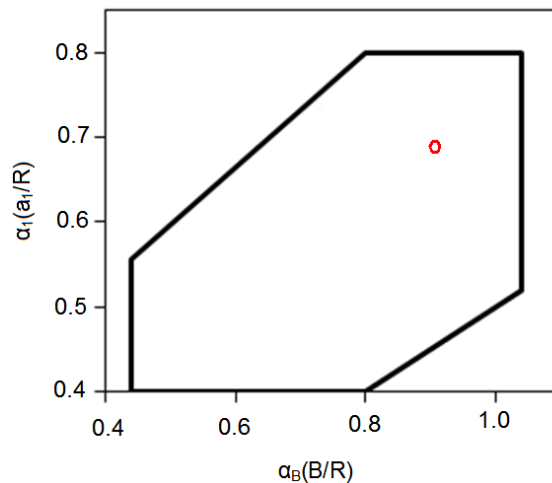


Figure 3.7 Valid geometrical ranges for CCNBD specimen

A circular 40 mm diamond saw was used to cut the required notch (Figure 3.8a). A specially designed jig recommended by ISRM was used to ensure that the chevron notches are exactly in the

centre of the disc. Additionally, the jig was controlled by an electronic alignment device that varies displacement along the x, y and z coordinates to control the centre alignment and penetration of the saw (Figure 3.8b). Subsequently, the saw was mounted to the machine and the electronic alignment device was set to zero at this level. Then the specimen was fixed and the saw was lowered to the required digitally read notch depth, and a jig was tied at this distance to the centre of the saw, to determine the centre of the notch end. Then a chevron notch was machined to one face. Next, the jig was adjusted again for the second face and the notched face of the specimen was mounted on the knife piece in order to ensure that the notches cut in both sides aligned properly with each other. The second chevron notch was machined in the specimen, and the specimen was ready to fracture test. Prepared samples are shown in Figure 3.9.

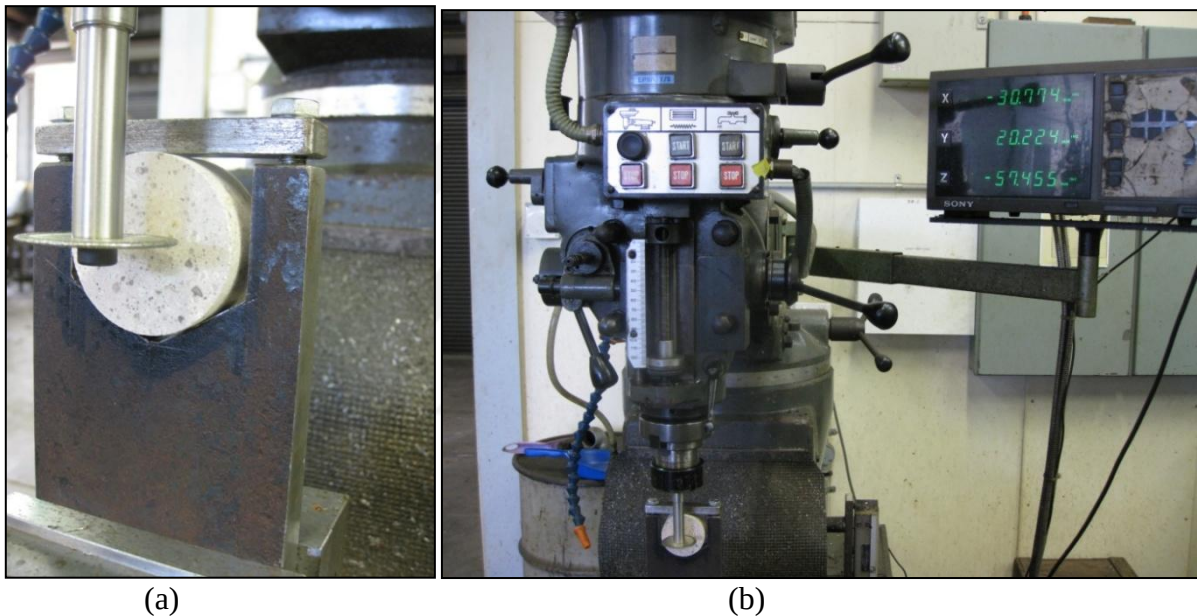


Figure 3.8 CCBND specimen preparation equipment (a) diamond saw (b) electronic alignment equipment



Figure 3.9 Prepared CCBND specimens

3.3 Tested Rocks

Bulk Brisbane tuff was used in both static and cyclic loading tests. Limited tests were carried out on sandstone and granite to compare to the results from the Brisbane tuff. Brisbane Tuff was chosen for several reasons: it is a massive rock type with no bedding (being an ash deposit) and it is easy to handle and prepare for testing, which would minimise time constraints for this research project. Moreover, Brisbane Tuff's massive character gives less test result variability, and it is a targeted rock type in the Brisbane area for its strength and stability properties in regard to tunnelling and excavation. Two types of tuff were found in the CLEM7 tunnel host rocks; welded tuff and stratified nonwelded tuff or altered tuff. Brisbane welded tuff is a fine-grained, massive rock of rhyolitic composition with coarser grains imparting a porphyritic texture. Quartz and feldspar phenocrysts 1–3 mm in size are embedded in a matrix consisting of polycrystalline silica. Small vesicles are often present. The welded tuff is light to dark grey when fresh, but can be pink, purple, white, green or orange in colour with some brown iron-staining in the weathered state. In contrast, stratified nonwelded tuff is probably associated with faulting. Stratified nonwelded tuffs are generally light grey to cream in colour with a sandy texture.

In most experiments, samples of welded Brisbane tuff from two different cores, NST50 and NST207, were used. Thin sections of each sample were prepared for petrographic analysis. Petrographic images of thin sections are given in Figure 3.10 and Figure 3.11. Both tuff samples are composed largely of quartz and K-feldspar, with small amounts of siderite (Fe-carbonate) and zeolite minerals. In general, quartz and feldspar minerals are embedded separately in primary silica cryptocrystalline cement without interlocking. Cryptocrystalline texture in igneous rocks is a very fine aggregate of crystals, and in minerals means the individual crystals are too fine to be distinguished even under a petrological microscope. According to thin-section analysis, secondary carbonate cement replacing primary silica cryptocrystalline cement was observed. Moreover, primary feldspars were altered to clay minerals (mainly kaolinite and illite) in some areas. This kind of cement fills the voids in the primary silica cryptocrystalline cement. Thin-section studies showed further that secondary microcrystalline quartz cement occurs within the primary cryptocrystalline cement (Figure 3.11a) and in some quartz minerals placed in that microcrystalline quartz cement (Figure 3.11b).

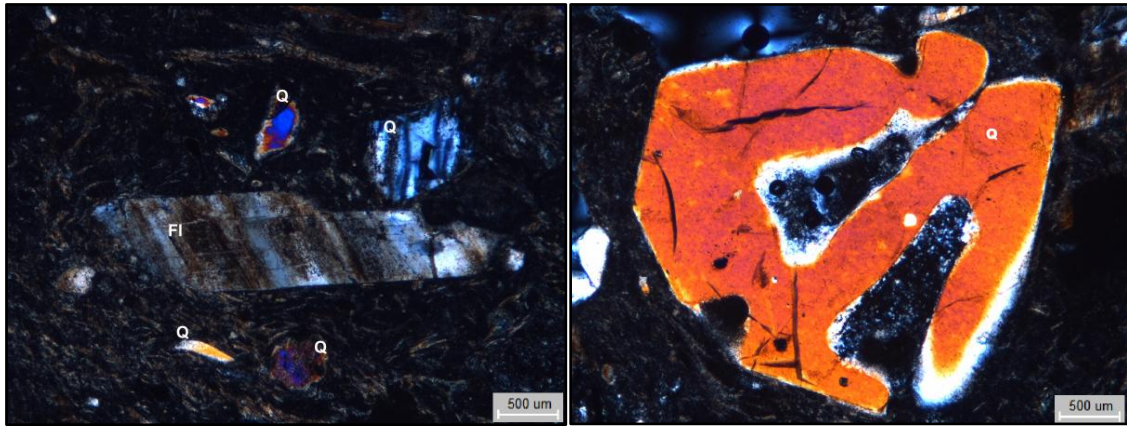


Figure 3.10 Photomicrograph of Feldspar (Fl) and Quartz (Q) mineral

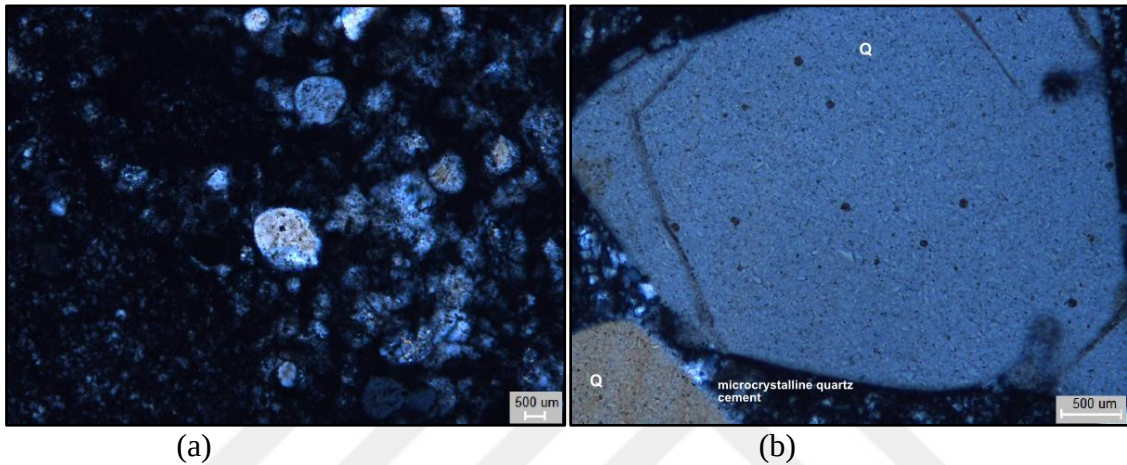


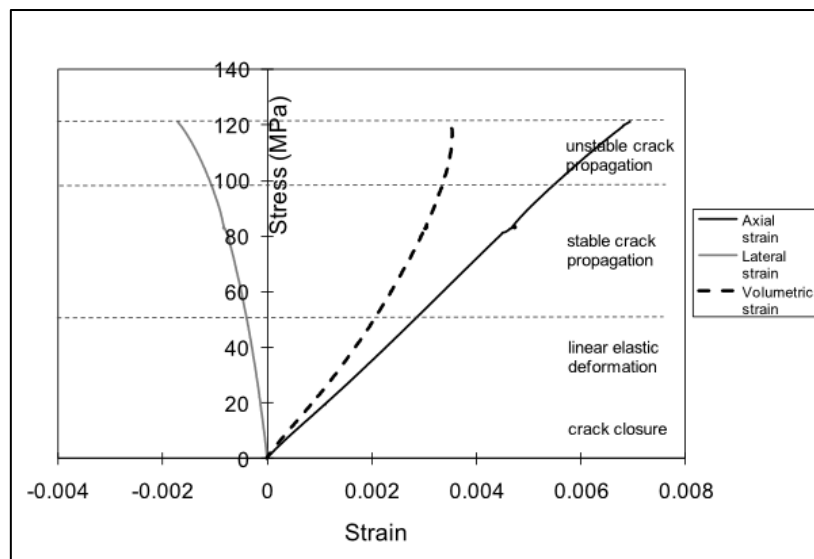
Figure 3.11 Microcrystalline quartz cement (a) and Quartz (Q) mineral in microcrystalline quartz and primary cryptocrystalline cement (b)

Some mechanical rock characterisation results supplied by different universities to the tunnel project team and specific results supplied from this research project are given in Table 3.1. It should be mentioned here that the results in Table 3.1 cover five different Brisbane tuff samples according to their formations and drill locations.

Table 3.1 Mechanical characterisation results for Brisbane tuff

Rock sample	Young's Modulus (GPa)	Poisson's ratio	UCS (MPa)	BTS (MPa)
Melbourne University	Max = 23.8 Min = 3.12	Max = 0.25 Min = 0.17	Max = 123 Min = 45	Max = 13.15 Min = 2.7
James Cook University	Max = 14 Min = 11	-	Max = 106 Min = 53	-
University of Queensland (This research)	Max = 25 Min = 19	Max = 0.26 Min = 0.22	Max = 205 Min = 97	Max = 15 Min = 8.0

UCS tests using strain-gauged cylindrical samples were used in those research studies to provide the most insight into delineating the stages of crack. Numerous researchers over the past 30 years have been studying fracture characteristics of laboratory-tested brittle rocks (Brace, 1961; Lajtai, 1974; Martin & Chandler, 1994; Tappointer & Brace, 1976). In general, these studies have shown that the failure process can be broken down into a number of stages based upon the stress–strain behaviour of a loaded rock specimen, including pre-existing ‘Griffith cracks’. These stages have been defined as crack closure, linear elastic deformation, stable crack propagation and unstable crack propagation. According to this definition, the relevant stages for Brisbane tuff are shown in Figure 3.12.

**Figure 3.12 Stress versus strain curve of a tested strain-gauged UCS sample**

Crack closure occurs during the initial stage of loading when pre-existing cracks, orientated at an angle to the applied load, close. During crack closure, the stress–strain response is nonlinear. Once the majority of pre-existing cracks have closed, linear elastic deformation takes place. Crack propagation at this point is considered as being stable, where controlling the applied load can stop crack growth. Then, unstable crack propagation starts and continues to the point where the numerous microcracks coalesce into larger cracks and the rock can no longer support an increase in load. This point is considered as the peak strength of the rock sample.

3.4 Scanning Electron Microscope

The SEM is a type of electron microscope that images the sample surface by scanning it with a high-energy beam of electrons in a raster scan pattern. The beam of electrons is generated by an electron gun, processed by magnetic lenses focused at the specimen surface and systematically scanned (rastered) across the surface of a specimen. The specimen is scanned in synchrony with the beam on the specimen in a one-to-one relationship between points on the specimen and points on the image viewing screen. The basic principles of the SEM can be summarised as follows:

Scanning = moving back and forth

Electron = uses a focussed electron beam

Microscopy = designed to enlarge an image

The SEM provides the competent user with an enormous advantages over the Light Microscope (LM) in three key areas:

1. Resolution at high magnification: the best resolution possible in a LM is about 200 nm, whereas a typical SEM has a resolution of better than 10 nm.
2. Depth of field: the height of a specimen that appears in focus in an image—more than 300 times the depth of field compared to the LM. This means that great topographical detail can be obtained.
3. Microanalysis: the analysis of sample composition, including information about chemical composition, as well as crystallographic, magnetic and electrical characteristics.

When the electron beam strikes a sample, these electrons will scatter through the sample within a defined area called the interaction volume. During the electron beam–specimen interactions, secondary products like Secondary Electrons (SE), Backscattered Electrons (BSE), X-rays, heat and

light will be formed. Image formation of surface structures using the SEM mainly depends on the production of secondary electrons. Secondary electrons are low-energy electrons and when produced deeper within the interaction volume, they are absorbed by the sample. Only secondary electrons close to the surface will be able to escape the specimen. In contrast, BSE electrons are high-energy electrons (> 50 eV) that have undergone simple or multiple elastic scattering events within the specimen.

A JEOL JMS-6460 LA brand SEM was used for rock fracture imaging in this research (Figure 3.9). JEOL JSM-6460 LA tungsten is a low-vacuum analytical SEM equipped with an integrated JEOL Hyper mini-cup Energy Dispersive X-ray Spectrometer (EDS). This EDS system has a polymer Ultra Thin Window (UTW), a SiLi crystal with a nominal 133 eV resolution, and integrated JEOL Analysis Station (V3.8) software used to collect and analyse the x-ray data. The structure of the JEOL JMS-6460 LA consists of: (1) external components to maintain the vacuum and other operating conditions; (2) a column that houses the electron optic system and includes the specimen chamber; and (3) a console on which the display screens and other control systems and keyboards are located (Figure 3.13).

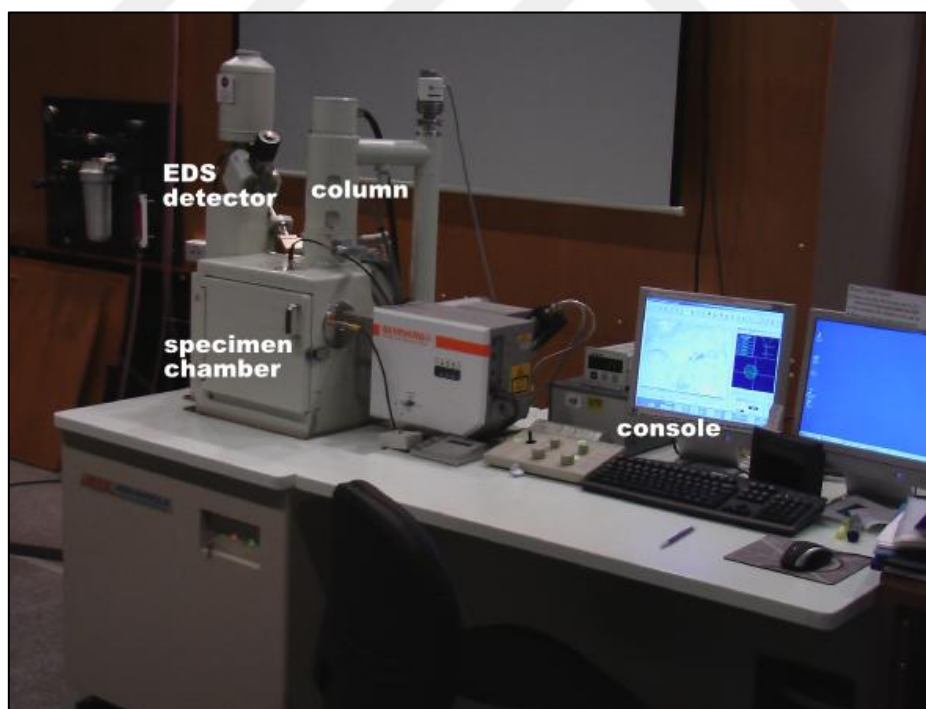


Figure 3.13 External components of the JEOL JMS-6460 LA SEM

In this study, all rock fracture images were obtained using Low Vacuum (LV) chamber pressures i.e., 1–50 Pa (with adjustable pressure between 10 Pa and 270 Pa). This allows certain samples to be observed uncoated and reduces damage to the specimens caused by the effects of a high vacuum.

LV mode uses only BSE to observe an image. This makes it more difficult to observe and focus, particularly at lower vacuum pressures and faster scan rates. Selecting the lowest scan rate and increasing the magnification were used to get better image quality and assist with focusing. The key to producing the best SEM image of a specimen is a full understanding of the effects of the machine variables. Some important variables are condenser lens current, stigmatism, working distance and accelerating voltage among others. One can make adjustments by changing these variables and often there is a trade-off between multiple variables. The best operational parameters of working distance, acceleration voltage and spot size, were determined after many preliminary tests. 15 kV acceleration voltage was used in this study with varying working distances between 8 mm and 14 mm.



CHAPTER 4

DETERMINATION OF TENSILE STRENGTH OF ROCKS— METHODS AND RESULTS

The tensile strength of rocks is much less than their compressive strength, thus, rock fracturing in rock engineering is mainly caused by relatively low tensile strengths and, hence, attention has been paid to the methods for measuring the tensile strength of rocks. Rock is a brittle material, and because all brittle materials are weak in tension the tensile strength of rocks becomes one of the most important parameters influencing deformability, fracture toughness and response to blasting. However, the tensile strength is usually measured in an indirect way, rather than directly, because of difficulties in implementing direct tension tests.

4.1 Methods

The difficulties associated with performing a direct uniaxial tensile test on a rock specimen have led to a number of indirect methods for assessing the tensile strength. The most popular of these indirect tensile tests consist of applying diametral compression between the platens of a compression testing machine to a rock cylinder that has thickness usually shorter than its diameter.

In rock mechanics, diametral compression tests (indirect Brazilian tests) and their shortcomings have received considerable attention, both theoretical and experimental, and it has become evident that they can never adequately replace the direct uniaxial tensile test, which is actually the basis for the definition of tensile strength. However, the indirect Brazilian test is particularly appealing for its simplicity in specimen preparation and testing, but it produces failure in a biaxial, rather than a uniaxial stress field. Hence, the terminology for tensile strength of rocks—indirect tensile strength or direct tensile strength—should be used correctly.

4.1.1 Direct tension test

Despite the importance of the tensile strength of rock in practice and in connection with theories of failure, direct measurements of tensile strength are difficult and not commonly made. Theoretically, the tensile strength of rock should be tested by direct tension methods, with the rock sample in a one-way tension stress state, and thus its ultimate stress can be taken as the rock tensile strength. In

direct tensile tests, the failure rate is relatively high because of the difficulty of making and clamping the rock sample.

Early measurements were made by cementing cylindrical specimens of rock into grips, which created stress concentrations at the grip in a manner analogous to those at the contact between specimen and platen in compression tests. Fairhurst (1964) overcame much of this difficulty by cementing the ends of the rock specimen with epoxy resin direct to a steel end piece having the same cross section as the specimen. These ends were attached to flexible cables, through which the tensile load was applied, so as to minimise the bending stress transmitted to the specimen.

The test in itself is very simple and short, although sample preparation requires a fairly long time and a skilful person. In general, stress concentrations near the ends of the samples are observed. Also, the failure plane in samples occurs at an angle to the axial direction of the sample, indicating shear stresses most likely due to misalignment. Another well-known important parameter in specimen preparation is that minor scratches on the surfaces of tensile specimens may have a considerable effect on their strength.

It is clear that the height/diameter ratio and the maximum aggregate particle size are related and affect the final results. The sample preparation technique (glue, alignment, etc.) is of paramount importance in order to ensure that the failure plane occurs far enough from the end caps and perpendicular to the sample axis.

Additional effort is needed in the following areas:

- specimen geometry (length/diameter ratio, maximum nominal aggregate size/diameter ratio)
- perpendicularity between end faces and axial direction
- sample configuration
- alignment device
- glue type
- amount of film thickness
- contact pressure between sample faces and end caps
- cure time
- testing temperature and load
- loading system capability suitable for the loading range used

There are two suggested international standard methods to determine direct tensile strength of rocks suggested by ISRM (1981) and ASTM (1989). Both methods are for direct tensile strength determination of intact cylindrical rock specimens.

In this study, a series of direct tension tests was carried out on Brisbane tuff. The diameter of direct tension test specimens was the same as that for the indirect tension test specimens. The direct tension test specimens were cylindrical core samples with a diameter of 52 mm and a length of 135 mm (a length:diameter ratio of 2.59).

In order to apply direct tension, two cylindrical steel caps were cemented to the ends of the specimens with Megapoxy PM high-strength gap-filling epoxy paste (Figure 4.1). The dimensions of the metal caps and specimen preparation were in accordance with ISRM (1981) standards. A linkage system was used to transfer the tensile load from the Instron loading frame to the specimen (Figure 4.1).

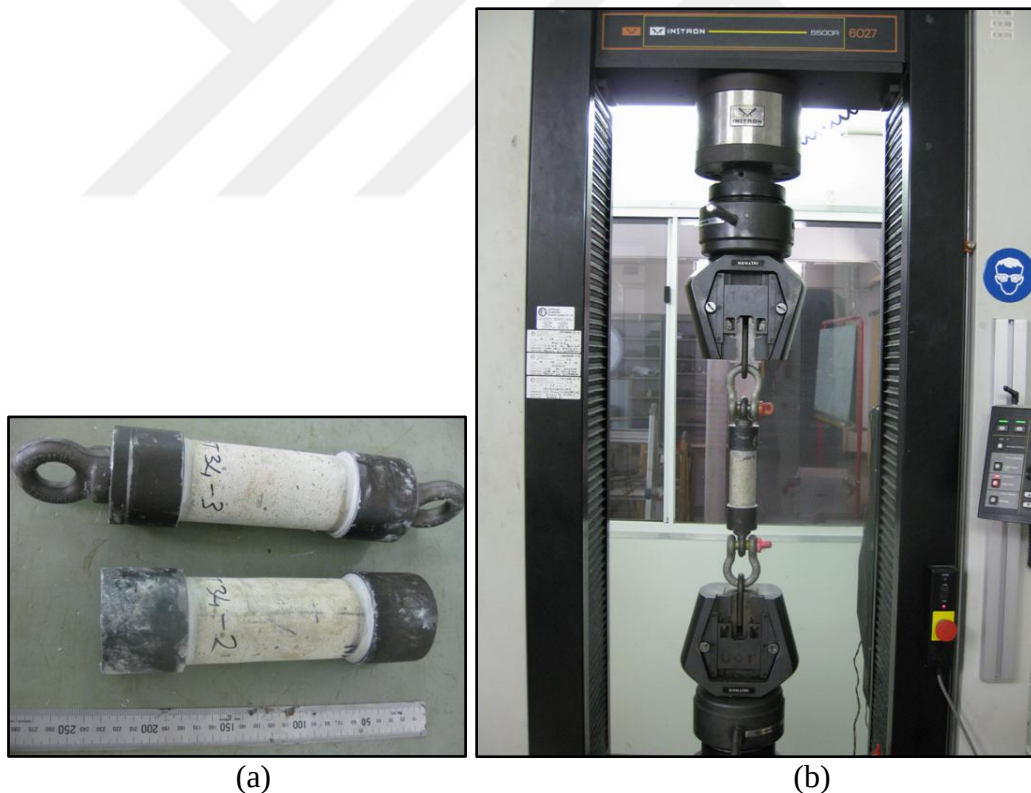


Figure 4.1 (a) Prepared specimens before direct tensile testing, (b) direct tensile loading test apparatus

Five replicate specimens were tested. However, torsion effects was observed in two test specimens due to misalignment of the load transfer system, and only the three successful replicate tests are reported. Results and test details are given in following sections.

4.1.2 ISRM suggested indirect Brazilian test

In 1978, the Brazilian test was officially proposed by the ISRM as a suggested method for determining the tensile strength of rock materials (ISRM, 1978). The Brazilian test, or splitting tension test, is performed by applying a concentrated compressive load across the diameter of a disc specimen. The justification for the test is based on the experimental fact that most rocks in biaxial stress fields fail in tension at their uniaxial tensile strength when one principal stress is tensile and the other finite principal stress is compressive with a magnitude not exceeding three times that of the tensile principal stress.

According to ISRM suggestions, two steel loading jaws are designed so as to contact a disc-shaped rock sample at diametrically opposed surfaces over an arc of contact of approximately 10° at failure (Figure 4.2). For the loading method, ISRM suggested that the load be applied via two steel loading jaws in contact with a disk-shaped rock specimen, and that the radius of the jaws be 1.5 times the specimen radius (ISRM, 1978).

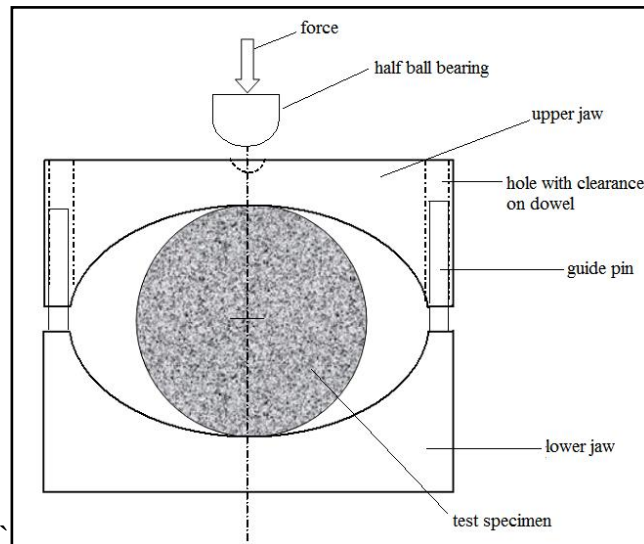


Figure 4.2 Loading jaws for indirect tensile Brazilian test

In a standard Brazilian test, the concentrated load is applied over an infinitesimally small width as a line load, but clearly this would lead to stresses of very high intensity. The tensile strength of a rock disc specimen σ_t is calculated using the equation:

$$\sigma_t = \frac{2P}{\pi Dt} \quad 4.1$$

where P is the failure load, and d and t are the diameter and thickness of the rock disc, respectively. Eq. (4.1) is based on the theory of elasticity for isotropic media. The formula gives the tensile stress perpendicular to the loaded diameter at the centre of the disc at the time of failure when the applied force is P .

In this study, a series of Brazilian disc tests was carried out using specimens prepared from Brisbane tuff, sandstone and granite. Most of the tests were carried out on Brisbane tuff, because it is a host rock of Brisbane's first motorway tunnel, CLEM7, from which core samples were obtained. Limited tests were carried out on sandstone and granite to verify that the results could be more generally applied than to a single rock type. Brisbane tuff behaves in an almost linear elastic manner for a significant portion of its axial stress–strain curve. The test specimens prepared were standard Brazilian discs with a diameter of 52 mm and thickness of 26 mm (a diameter:thickness ratio of 0.5). The load was applied by a stiff hydraulic Instron loading frame, with a loading rate suggested by ISRM of 200 N/s (ISRM, 1978) on to the standard loading jaws (Figure 4.3).



Figure 4.3 Brisbane tuff disc sample between standard Brazilian jaws

The tensile strength of the five rock specimens tested using the standard Brazilian jaws was calculated from the formula given by ISRM (1978).

4.1.3 Different angled loading arc tests

The classical theory assumes that the concentrated load is applied over an infinitesimally small width as a line load, but clearly this would lead to stresses of very high intensity. The actual loads are not concentrated but are distributed over a finite arc of the disc. The distributed load applied to a disc under diametral compression is more difficult to analyse than a concentrated load. Hondros (1959) analysed the Brazilian test for the case of a thin disc loaded by a uniform pressure, applied radially over a short strip of the circumference at each end of the disc. He obtained the full-field stresses in series solutions by using the series expansion technique and applied these solutions to evaluate the Young's modulus and Poisson's ratio of concrete by measuring strain.

The Brazilian test, or splitting tension test, is performed by applying a concentrated compressive load across the diameter of a disc specimen. Thus, stress concentration is also a problem in the Brazilian test when using standard Brazilian loading jaws. It is generally found with standard Brazilian tests that there are several fractures branching from the diametral plane, some of which appear to be wedges near the contacts. Although the Brazilian test has been studied extensively, several key questions remain unresolved: for example, how to guarantee crack initiation at the centre of the specimen (beneath the concentrated load), how to obtain an accurate representation of the tensile strength of the rock from the test, and how to obtain closed-form expressions for the complete stress and strain fields both for the Brazilian tests and for the case when the load is applied as a pressure acting normal to, and uniformly across, an arc of finite length.

In recent years, different geometries such as flattened Brazilian disks and loading arcs to eliminate stress concentrations at the loading points, have been proposed to determine the tensile strength of brittle materials (Wang *et al.*, 2003; Yong *et al.*, 2009).

In this study, the traditional Brazilian test has been modified in one aspect. The two steel loading jaws were replaced by two special loading arcs with various arc angles in order to reduce the stress concentrations near the loading points and to get a centre crack in the specimen. The loading arcs are in perfect contact with the specimen (i.e., they have the same radius as the specimen; Figure 4.4). Three series of indirect tension tests were conducted, with: (1) 15° steel loading arcs, (2) 20° steel loading arcs, and (3) 30° steel loading arcs. The steel loading arcs were machined from standard mild steel, as recommended by ISRM (1978).

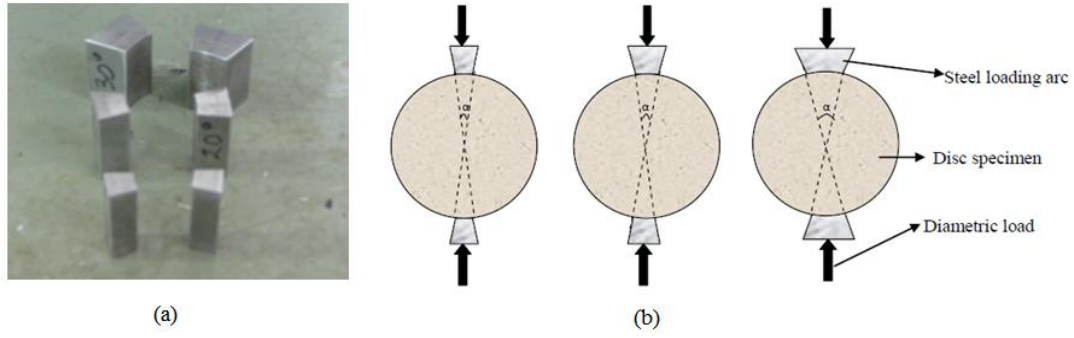


Figure 4.4 (a) Steel loading arcs (b) diagram of steel loading arcs and disc specimens

The test specimens were standard Brazilian discs with a diameter of 52 mm and thickness of 26 mm (a diameter:thickness ratio of 0.5). The load was applied by a stiff hydraulic Instron loading frame, with a suggested loading rate of 200 N/s (ISRM, 1978). Five replicate specimens were tested.

4.2 Results

Both direct and indirect tensile strength tests were performed on Brisbane tuff and two other rock type specimens. A total of 5 samples for the direct tensile strength test and 35 samples for the indirect tensile strength tests, including steel loading arc tests, were analysed.

In this study, the main focus of the indirect tensile strength tests was to achieve tensile failure due to a central crack along the vertical diameter of the disc, which is assumed to be the region of maximum indirect tensile stress. To further investigate the role of diametral load in contributing to the determination of indirect tensile strength, a series of indirect tension tests using 30°, 20° and 15° loading arcs and standard Brazilian jaws were performed on disc specimens of two other rock types: sandstone and granite.

4.2.1 Direct tensile strength

The direct tensile strength of a specimen can be calculated by dividing the maximum load applied to the specimen by the original cross-sectional area. However, torsion effects was observed in two of the five replicate test specimens due to misalignment of the load transfer system, and only the three successful replicate tests are reported. These results are given in Table 4.1.

Table 4.1 Results of direct tensile tests on Brisbane tuff

Specimen	Recorded maximum load (kN)	Direct tensile strength (MPa)
Replicate 1	13.2	6.22
Replicate 2	12.2	5.74
Replicate 3	10.23	4.98
Average	11.88	5.65

The alignment and gluing took approximately 3 hours. Typically, only one or two samples can be tested a day if only one jig is available; because of this, the experiment was time consuming. The capping operation is the most complicated. As previously mentioned an alignment jig was tailor-made for gripping the samples and keeping the end faces perpendicular to the specimens' axial direction. Stress concentrations near one of the ends continued to occur. Shear failure, probably due to misalignment, was the common factor encountered in the samples tested as shown in Figure 4.5 clearly.

**Figure 4.5 Tested samples (a) before caps are removed (b) after caps are removed**

Figure 4.6 shows the stress–strain plots of the direct tension tests. It is clear that there is not good consistency between tests in terms of stress–strain behaviour and failure load.

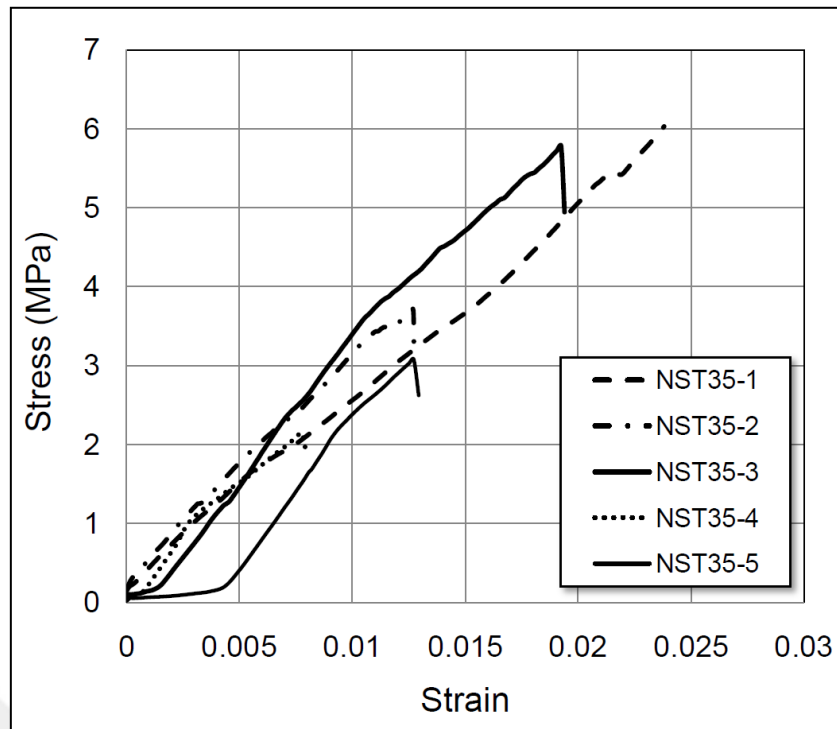


Figure 4.6 Stress–strain plots of direct tension tests

It was decided after the testing that the alignment device should be reviewed to check the parallelism of the end faces and their alignment with regard to the axial direction of the sample, in order to obtain failures in the middle of the specimens and consistently away from the end caps. Sample preparation seems to be the key to many of the problems encountered. Special care should be taken in cutting the ends and in gluing the specimen to the end plates. It is suggested that a minimum amount of epoxy resin be used that is compatible with the required stress and that a uniform compressive pressure be applied on the end caps to obtain a uniform film of epoxy binder. Finally, from the type of failure seen in many of the samples in which shear stress is evident (failure at the end caps), it was determined that a check of the alignment device was needed.

4.2.2 Indirect Brazilian tensile strength

A series of Brazilian disc tests was carried out on specimens of Brisbane tuff, sandstone and granite. Most of the tests were carried out on Brisbane tuff, because it is a host rock of Brisbane's first motorway tunnel, CLEM7, from which core samples were obtained. Limited tests were carried out on sandstone and granite to verify that the results could be more generally applied than to a single rock type. Brisbane tuff behaves in an almost linear elastic manner for a significant portion of its axial stress–strain curve. The test specimens prepared were standard Brazilian discs with a diameter of 52 mm and thickness of 26 mm (a diameter: thickness ratio of 0.5). The load was applied by a stiff hydraulic Instron loading frame, with a loading rate suggested by ISRM (1978) of

200 N/s. In order to improve the contact condition between the loading arcs and the disc specimens, disc specimens were covered with adhesive paper stripe as recommended by ISRM (1978). On the other hand, it is believed that the value of frictional stress (if there is) is very small when compared by produced tensile stresses in the centre of disc to neglect. To calculate the tensile strength from Brazilian test, one must know the principal tensile stress, in particular at the rock disk centre, where a crack initiates. No tangential stress is to be expected when the friction is zero. But this is never the case in real tests (Lavrov and Velvort, 2002). However this indication is for the boundary value problem at rock-platen contact. It may be possible for line contact situation, standard Brazilian test, where the crack initiation start. Also, it was concluded in same research that in the case when the tangential stress is directed inwards (a different ratio between rock's and platen's elastic properties (rock softer than the platen) would produce tangential stresses directed inwards., tangential load can produce higher concentration of compressive circumferential stress near the boundary, but even in this case the state of stress in the interior of the disk is as good as unchanged (Lavrov & Velvort, 2002). This means that a friction force does not induce any ambiguity in the tensile strength value obtained from Brazilian test, provided that fracture initiates in the central part of the disk. Whether the tangential stress is acting in- or outward, should depend on the elastic properties of disk and platens (or inserts) and probably on the friction conditions.

The tensile strength of the rock specimens tested using the standard Brazilian jaws were calculated using Eq. (4.1), the formula given by ISRM (1978). The details of the tests and test results for two different Brisbane tuff specimens, NST35 and NST207, are given in Table 4.2.

Table 4.2 Results of indirect tensile tests on Brisbane tuff disc specimens

	Recorded maximum load (kN)		Indirect tensile strength (MPa)	
	NST35	NST207	NST35	NST207
Replicate 1	26.29	24.58	12.36	11.56
Replicate 2	24.39	19.45	11.47	9.15
Replicate 3	25.42	20.98	11.96	9.87
Replicate 4	23.49	21.86	11.05	10.28
Replicate 5	25.97	25.06	12.22	11.79
<i>Average</i>	<i>25.11</i>	<i>22.38</i>	<i>11.81</i>	<i>10.53</i>

Two typical stress–strain curves of standard Brazilian tests are shown in Figure 4.7.

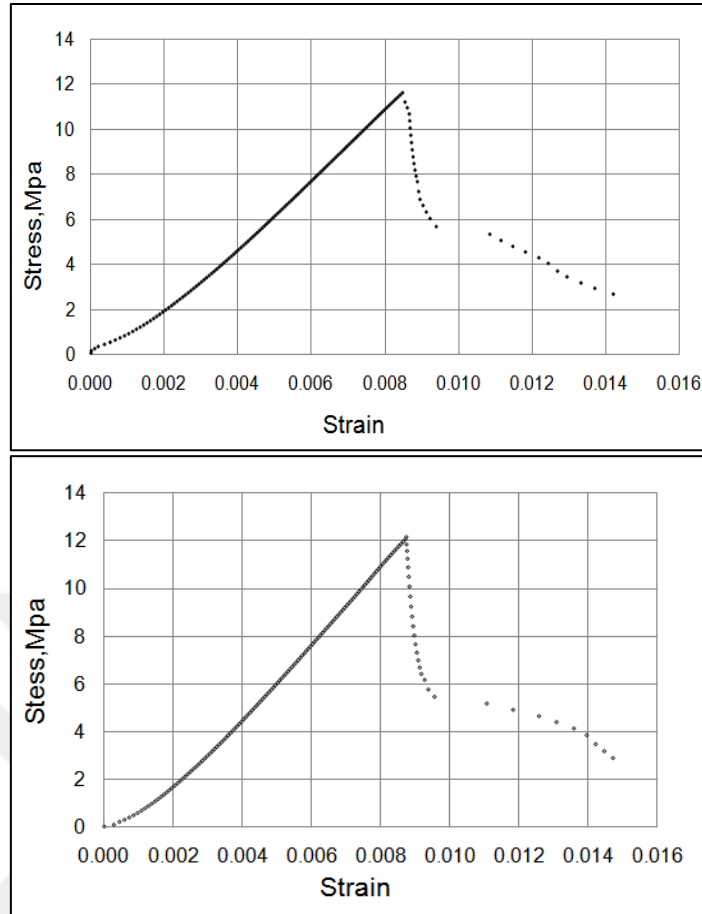


Figure 4.7 Stress–strain curves of standard Brazilian tests

In the present tests on Brisbane tuff, loading with the Brazilian jaws caused catastrophic crushing failure of the disc specimens (Figure 4.8). The occurrence of several fractures branching from the diametral plane gives rise to doubt about the mechanism of failure in this test. It is likely that these multiple fractures were caused by the release of energy stored in the testing machine after the specimen split into two hemi-cylinders, making the standard Brazilian test very sensitive to the stiffness of the testing machine used.



Figure 4.8 Failed Brisbane tuff specimens (a) NST35 and (b) NST207 under standard Brazilian jaws

Because the contact forces between disc and jaws cannot be considered as a line, as explained in previous sections, the failure of the sample into several cleavage planes instead of two halves proves that the indirect tensile strength from the failure load obtained by the standard Brazilian test might not represent the actual tensile strength of brittle rocks.

4.2.3 Indirect tensile strength by loading arcs

In this study, the main focus was to achieve tensile failure due to a central crack along the vertical diameter of the disc, which is assumed to be the region of maximum indirect tensile stress. Three series of indirect tension tests were conducted, with the following parameters: (1) 15° steel loading arcs, (2) 20° steel loading arcs, and (3) 30° steel loading arcs. Up to four repetitions were carried out.

Because the loading geometry of steel arcs is different from standard Brazilian jaws, the formula given by ISRM (1978) cannot be used to calculate the indirect tensile strength of specimens under loading arcs. In this study, the tensile strength of disc specimen under loading arcs was not calculated at the centre of the disc by using Hondros (Hondros, 1959) formula, because the formula proposed by ISRM was derived from Hondros's (Hondros, 1959) solution for stresses at the centre of the disc. That's why, indirect tensile strength of disc specimen under loading arcs were calculated at the very near of centre of the disc by using Eq. 4.2.

$$\sigma_{\theta} = -\frac{2p}{\pi} \left[\frac{(1 - r^2/R^2)\sin 2\alpha}{\left(1 + \frac{2r^2}{R^2\cos 2\alpha} + r^4/R^4\right)} + \tan^{-1} \frac{(11r^2/R^2)}{(1 + r^2/R^2)} \tan \alpha \right] \quad 4.2$$

where is σ_{θ} the tangential normal stress at a point situated at r and θ , p is applied stress, r is radial distance of a point from the origin, R is the radius of the disc and α is the half central angle of the applied distributed load.

To further investigate the role of diametral load in contribution to the determination of indirect tensile strength, a series of indirect tension tests using 30°, 20° and 15° loading arcs and standard Brazilian jaws were performed with two different rock types: sandstone and granite. The details of the tests and test results are given in Table 4.3. The maximum recorded ultimate load was obtained using a loading arc with $2\alpha = 30^\circ$ in all rock types.

Table 4.3 Results of indirect tensile tests using loading arcs

Specimen	Recorded maximum load (kN)			Recorded maximum load (kN)					
	Brisbane tuff (NST35)			Sandstone			Granite		
	15° loading arc	20° loading arc	30° loading arc	15° loading arc	20° loading arc	30° loading arc	15° loading arc	20° loading arc	30° loading arc
Rep-1	12.50	17.06	21.10	8.61	10.43	12.15	33.93	38.98	46.54
Rep-2	16.39	19.82	24.60	9.10	10.51	11.72	32.98	39.73	47.03
Rep-3	15.65	20.23	21.13	-	-	-	-	-	-
Rep-4	14.70	19.41	22.17	-	-	-	-	-	-
Average	14.81	19.20	22.30	8.95	10.47	11.91	33.50	39.40	46.75

The tested Brisbane tuff samples under loading arcs are shown in Figure 4.9. A single failure plane diverged from the load axis, with secondary cracks under 15° loading arcs (Figure 4.9a). Different from the results of 15° loading arcs, one axial splitting failure plane through the diametral loading axis was obtained with the samples tested using the 20° loading arc (Figure 4.9b). Unlike the results obtained by standard Brazilian tests and the 15° and 20° loading arc tests, arrested and vertical aligned central cracks were obtained with 30° loading arc tests (Figure 4.9c).



(a)



(b)



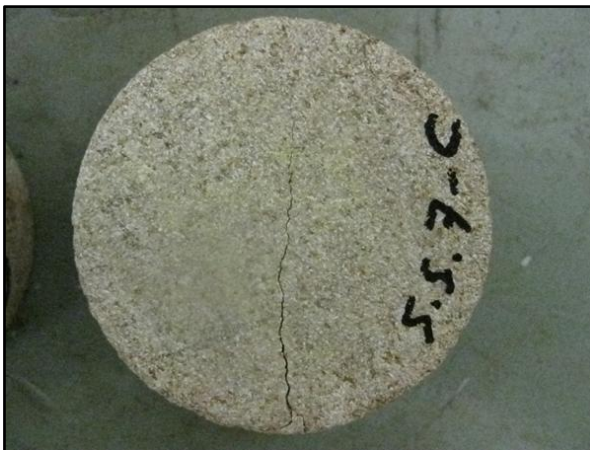
(c)



(d)

Figure 4.9 Failed Brisbane tuff samples under (a) Brazilian jaws (b) 15° loading arc (c) 20° loading arc (d) 30° loading arc

Examination of failed sandstone specimens showed that the standard Brazilian tests did not cause violent failure in sandstone samples because sandstone is a more porous and less brittle rock compared with Brisbane tuff and granite (Figure 4.10a).



(a)



(b)



(c)

(d)

Figure 4.10 Failed sandstone samples (a) standard Brazilian jaws (b) 15° loading arc (c) 20° loading arc (d) 30° loading arc

The effect of type of loading on tensile damage in a disc centre by using loading arcs and standard Brazilian jaws was observed easily in granite samples (Figure 4.11 a, b, c and d) as in Brisbane tuff. An arrested central crack was observed in the granite samples from the 30° loading arc tests. However, the crack at the centre of the granite sample was closed and almost invisible (Figure 4.11d) compared with the Brisbane tuff samples. In general, any crushing at the loading points was not observed with all type of tests.



(a)



(b)

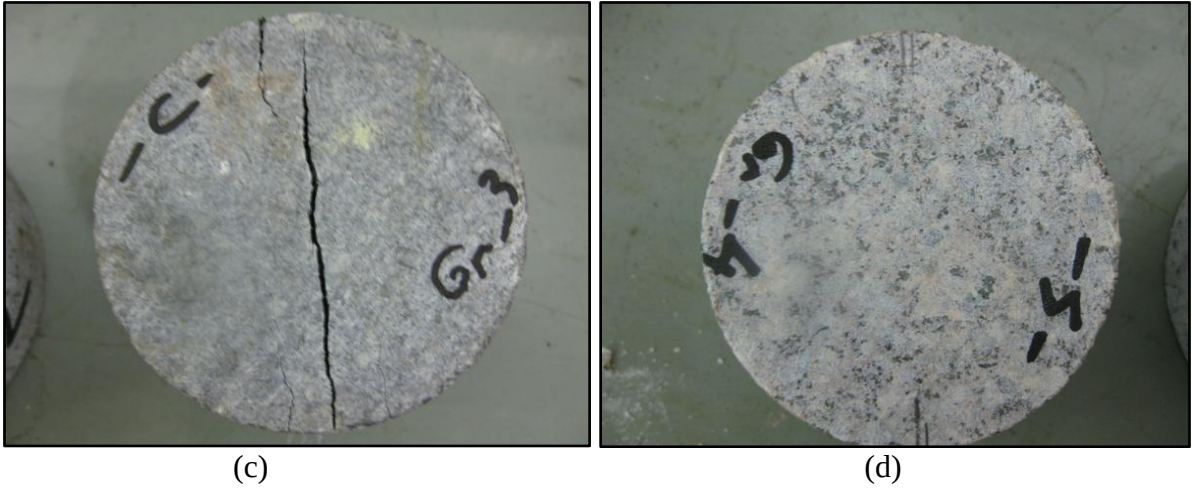


Figure 4.11 Failed granite samples under (a) standard Brazilian jaws (b) 15° loading arc (c) 20° loading arc (d) 30° loading arc

From this series of experimental studies, it was observed that the ultimate failure load increased with increasing angle of the loading arcs with less strain. Stress values for standard Brazilian tests were calculated by using Eq. (4.1), and the stress values for loading arcs were calculated using Eq. (4.2). Figure 4.12 schematically shows the stress–strain relation in a comparison between the standard Brazilian test and the loading arc tests. The highest stiffness was obtained with the 30° loading arc with hardly any plastic deformation in all loading geometries. In contrast, there is a slightly concave upward curve at the beginning of the stress–strain plot of the standard Brazilian tests. This shows that a slight hysteresis may be observed, but loading and unloading at that load level cause no irreversible changes. This nonlinear behaviour at the beginning of the stress–strain plot again shows that the accuracy of the standard Brazilian test is uncertain. Similar nonlinear behaviours at the beginning of stress–strain plots of Brazilian tests are shown in Figure 4.7.

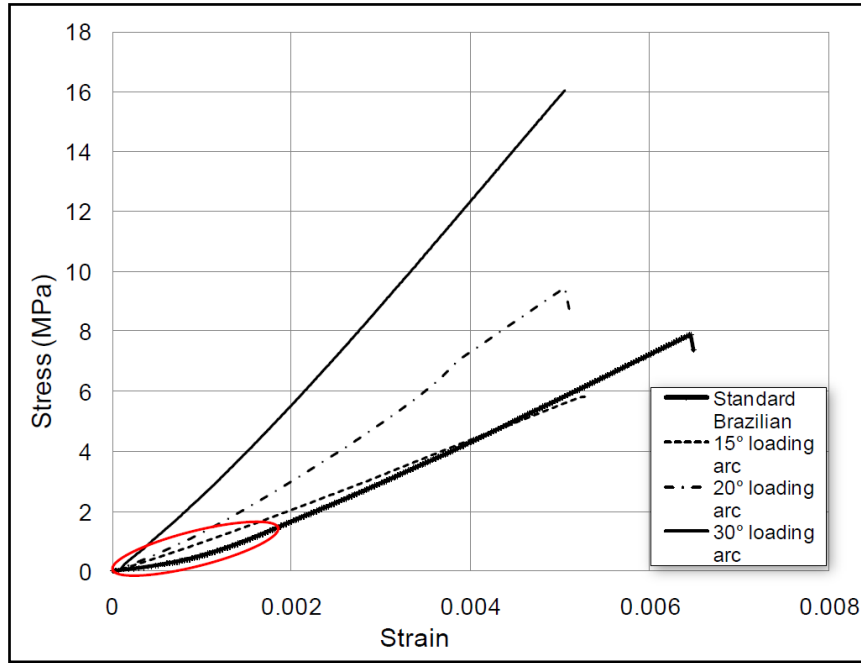


Figure 4.12 Comparison of stress–strain plots for the standard Brazilian and the loading arc tests

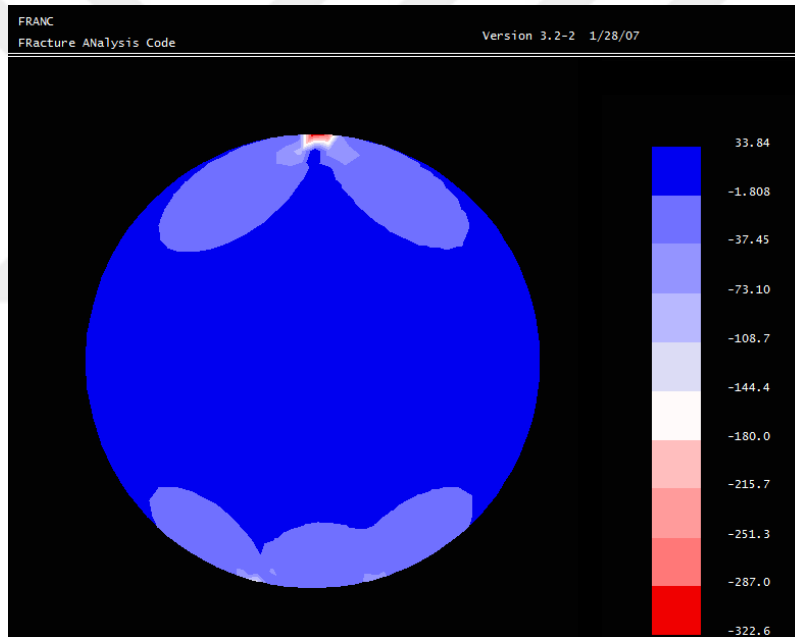
4.2.4 Numerical modelling results

A series of finite element analyses were conducted to understand stress distribution in a disc specimen under different loading types. 2D Finite Element Method (FEM) modelling was done using FRANC2D (FRacture ANalysis Code). In numerical models, the geometry of the sample has been assumed as a continuous, isotropic and homogeneous elastic body with 52 mm radius. In simulations, the rock has a Young's modulus of 22 GPa and a Poisson's ratio of 0.24. Load is expressed as traction pressure, which corresponds to a projected width of the loaded section of the disc, and was applied uniformly on the projected width considering the contact area between loading arcs and rock samples during the tests. In all simulations, the bottom of the disc was fixed in both x and y directions with the same width of the loading traction under the steel arcs. It should be noted that the loading traction of the standard Brazilian tests was not considered a line load because the 'point load' option of FRANC2D was not used. Thus, load was applied on a 1–2 mm traction width by assuming a distributed load with $2\alpha = 2^\circ$ in standard Brazilian modelling. Because the loading arcs and Brazilian jaws were not simulated, the effect of friction between the loading plates and the rock was eliminated.

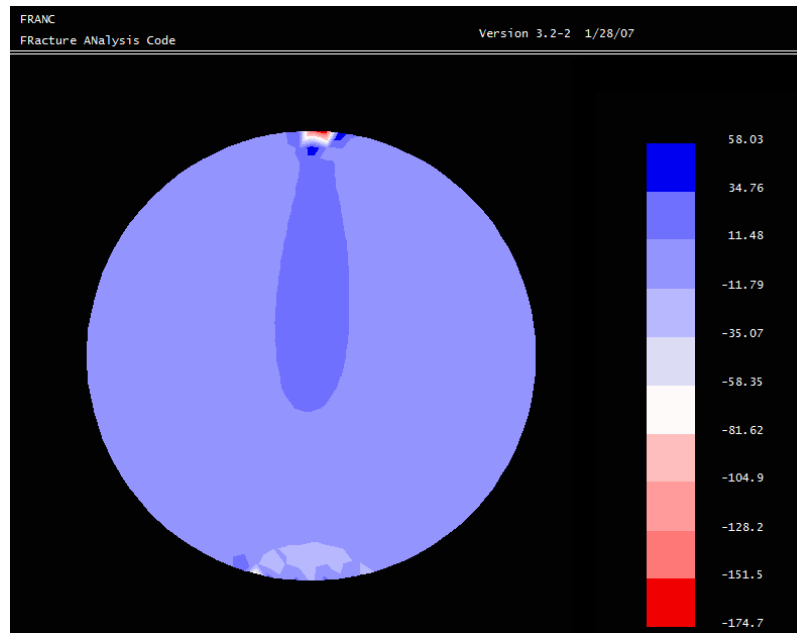
Because stresses along the loading diameter are the most concern, the tensile stress component (σ_x) in a specimen under different loading models is shown in Figures 4.13, 4.14, 4.15 and 4.16. In all figures, it can be seen that local maximum compressive stresses developed near the boundary and

distributed under the loading traction. However, this phenomenon disappeared far from the force acting point. In general, it was found that there is a concentrated tensile zone between the loading boundary and the fixed bottom in all simulations. However, a particular central concentrated tensile zone was obtained with $2\alpha = 30^\circ$ loading arc simulations (Figure 4.16).

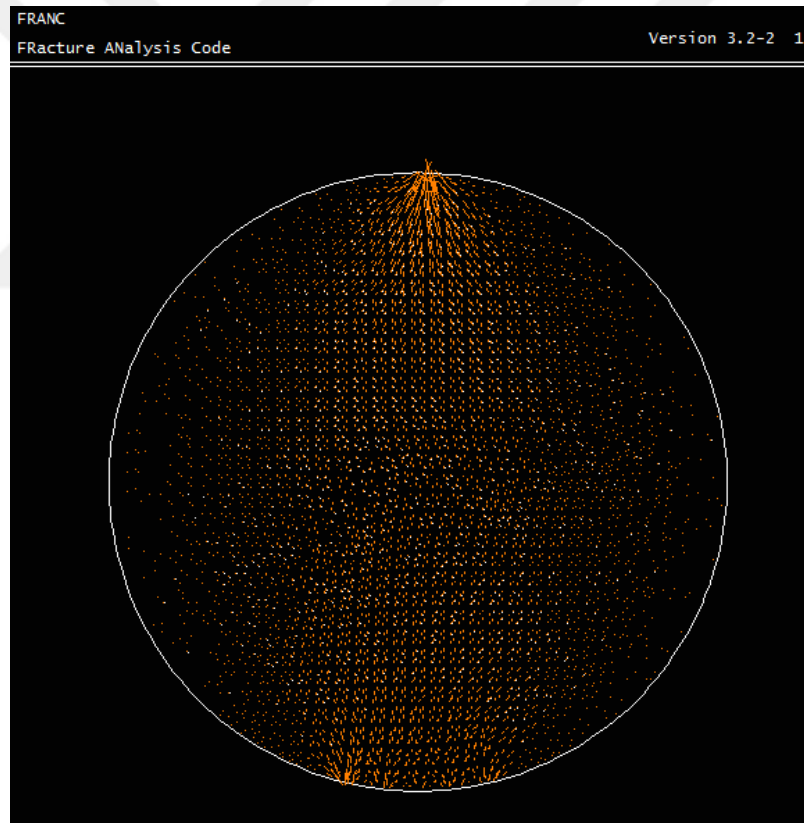
The highest value compressive load took place along a narrow strip in the standard Brazilian simulations (Figure 4.13a). In contrast, the highest tensile stress zone is closer to the load boundary when compared with the tensile zones obtained in loading arc simulations (Figure 4.14a, b, Figure 15a, b and Figure 16a, b). This may help to explain why tensile cracks tend to start from the loading boundary with standard Brazilian tests. Moreover, the highest maximum principal stress under standard Brazilian simulations was tensile, only took place under the compressive zone and was mostly effective along the upper half of the disc (Figure 4.13b).



(a)

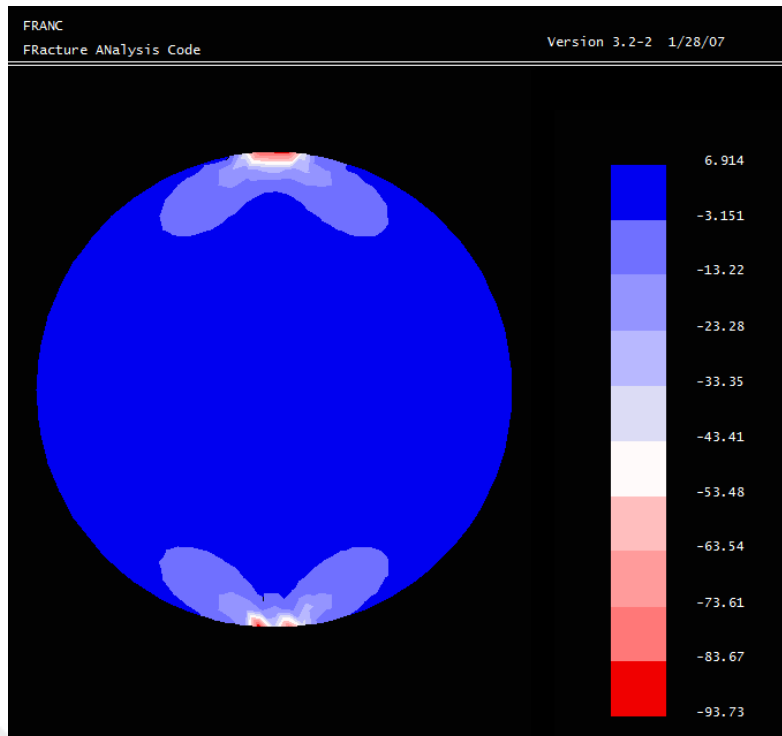


(b)

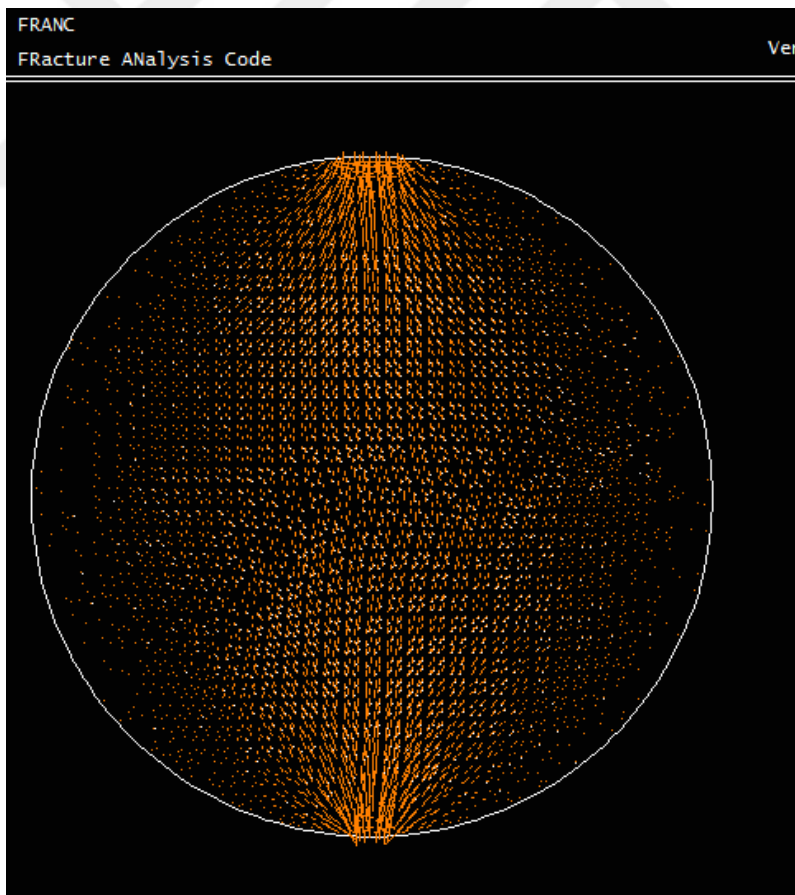


(c)

Figure 4.13 Standard Brazilian jaw simulations (a) Horizontal stress distribution (MPa) (b) Maximum principal stress distribution (+: tensile, -: compressive) c) Tensile (white) and compressive (orange) stress bars

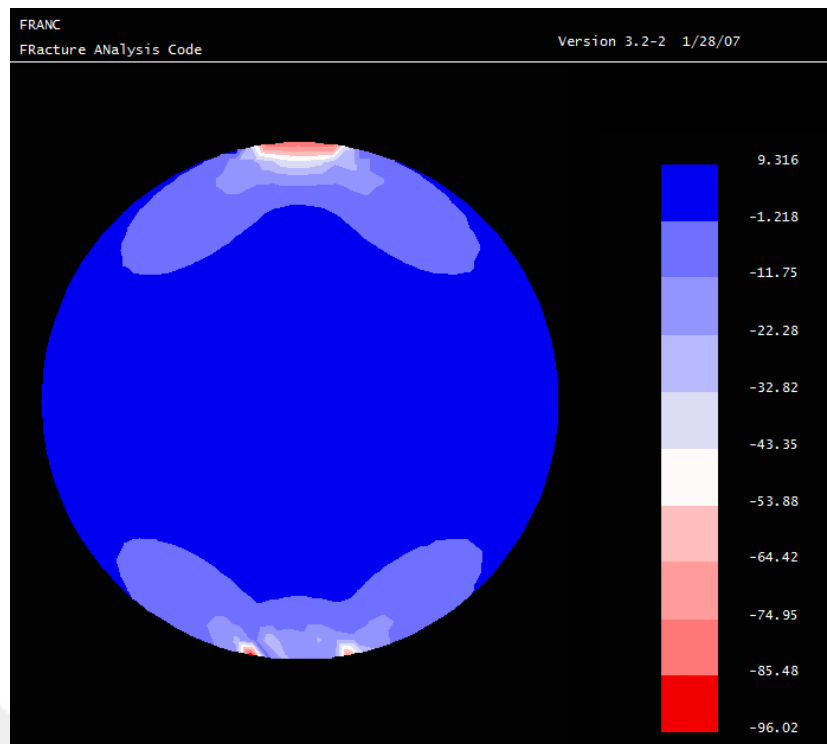


(a)

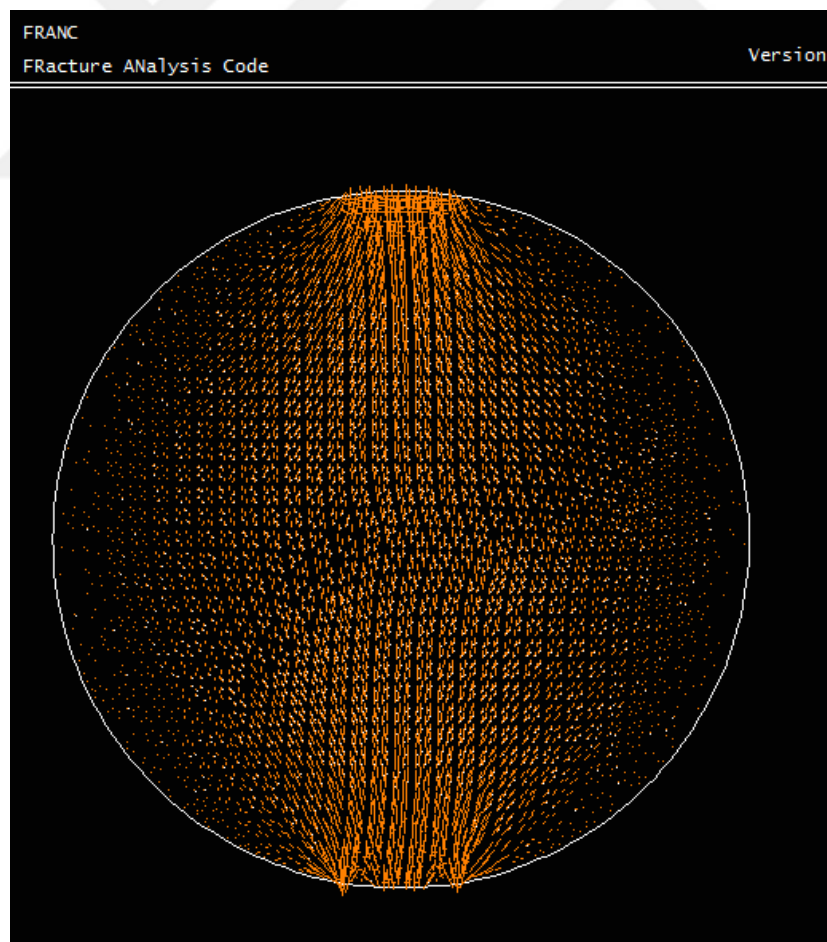


(b)

Figure 4.14 15° loading arc simulations (a) Horizontal stress distribution (MPa) (+: tensile, -: compressive) (b) Tensile (white) and compressive (orange) stress bars

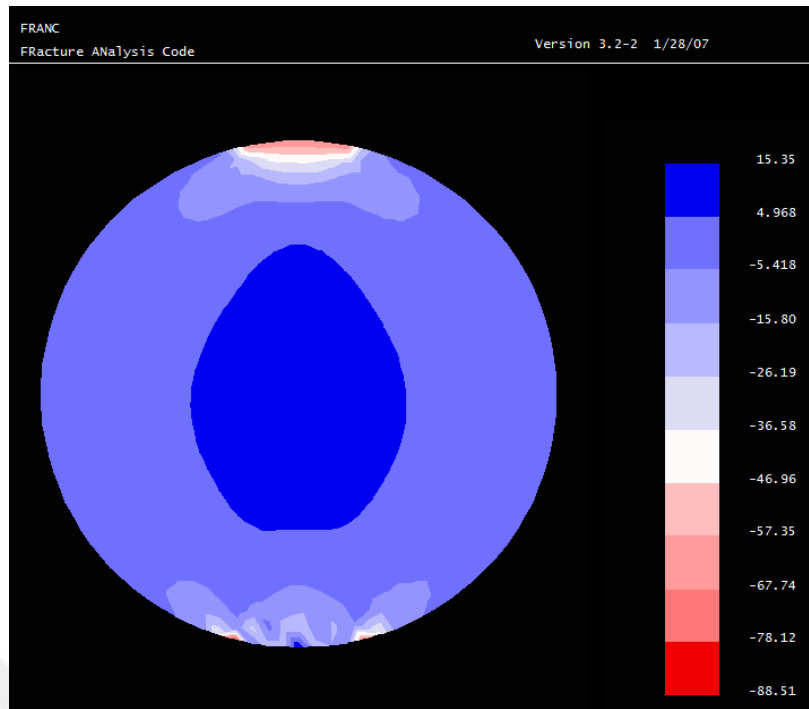


(a)

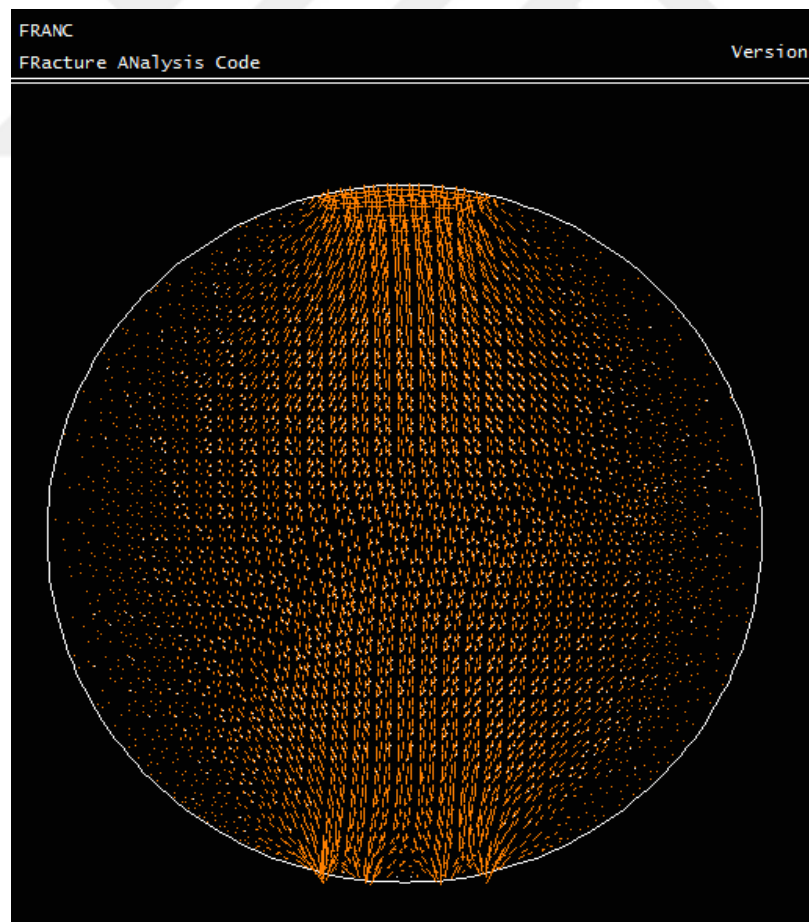


(b)

Figure 4.15 20° loading arc simulations (a) Horizontal stress distribution (MPa) (+: tensile, -: compressive) (b) Tensile (white) and compressive (orange) stress bars



(a)



(b)

Figure 4.16 30° loading arc simulations (a) Horizontal stress distribution (MPa) (+: tensile, -: compressive) (b) Tensile (white) and compressive (orange) stress bars

Figure 4.17 shows the tensile stress distribution along the horizontal diameter (AX) under standard Brazilian concentrated load and loading arcs. The maximum concentrated tensile stress took place at the centre of the disc under standard Brazilian concentrated load. In contrast, the minimum tensile stress concentration at the centre of the disc was obtained with $2\alpha = 15^\circ$ loading arc simulations. It is seen that the tensile stress increases and tensile zone is confined to a narrower region along the vertical direction when 2α increases in loading arc simulations (Figure 4.17).

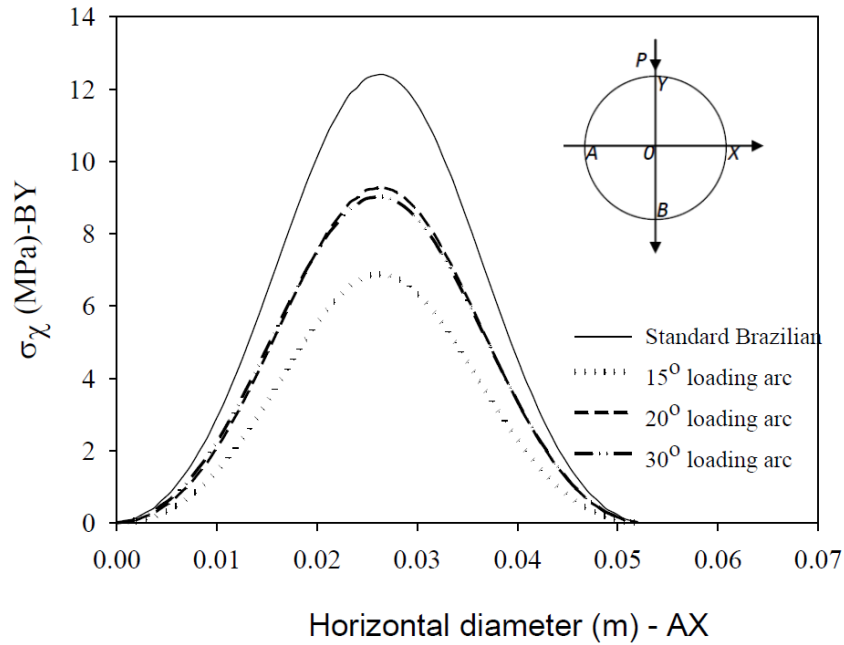


Figure 4.17 Tensile stress distributions along horizontal diameter of the disc

4.3 Discussion of Tensile Strength Results

The starting point for a study of failure in Brazilian discs under diametral compression is determination of the stress field, assuming that the material is homogeneous, isotropic and linearly elastic. In reality, however, many brittle materials such as rocks have nonlinear stress–strain characteristics because of pre-existing cracks, pores, etc.

According to the Griffith criterion (Griffith, 1920) used for failure in solid discs, the exact centre of the disc is the only point at which the conditions for tensile failure at a value equal to the uniaxial strength are met. However, in the determination of indirect tensile strength experiments described herein, the disc specimens shattered into several pieces under the standard Brazilian jaws. In contrast, applying distributed diametral loads with loading arcs causes single failure planes along the loaded diameter in all three rock types tested. It is clear that the geometry of loading

configuration, loading direction and sample geometry are related and affect the tensile strength of rocks.

4.3.1 Comparison between direct and indirect tensile strengths

Theoretically, the tensile strength of rock should be tested by the direct tension method, with the rock sample in a one-way tension stress state. However, the Brazilian test, or splitting tension test, is performed by applying a concentrated compressive load across the diameter of a disc specimen. The justification for the test is based on the experimental fact that most rocks in biaxial stress fields fail in tension at their uniaxial tensile strength when one principal stress is tensile and the other finite principal stress is compressive with a magnitude not exceeding three times that of the tensile principal stress. Thus, making a comparison between direct tension test and indirect tension test results will be meaningless due to the different loading geometries and relative stress states in the specimens.

The tensile strength of Brisbane tuff was found to be 5.65 MPa with direct tension tests. In contrast, the indirect tensile strength of Brisbane tuff was found to be almost double that of the direct tensile strength, 11.8 MPa with the standard Brazilian test. This result clearly shows the difference between direct and indirect tensile strength testing of rocks.

4.3.2 Analytical and numerical studies

Hondros (1959) analysed this problem with the Brazilian test for the case of a thin disk loaded by a uniform pressure, applied radially over a short strip of the circumference at each end of the disc. He obtained the full-field stresses in series solutions by using the series expansion technique as follows:

$$\begin{aligned}\sigma_r &= -\frac{2p}{\pi} \left\{ \alpha + \sum_{n=1}^{n=\infty} \left[1 - \left(1 - \frac{1}{n} \right) \left(\frac{r}{R} \right)^2 \right] \left(\frac{r}{R} \right)^{2n-2} \sin 2n\alpha \cos 2n\theta \right\} \\ \sigma_\theta &= -\frac{2p}{\pi} \left\{ \alpha - \sum_{n=1}^{n=\infty} \left[1 - \left(1 + \frac{1}{n} \right) \left(\frac{r}{R} \right)^2 \right] \left(\frac{r}{R} \right)^{2n-2} \sin 2n\alpha \cos 2n\theta \right\}\end{aligned}\tag{4.3}$$

$$\tau_{r\theta} = -\frac{2p}{\pi} \left\{ \sum_{n=1}^{n=\infty} \left[1 - \left(\frac{r}{R} \right)^2 \right] \left(\frac{r}{R} \right)^{2n-2} \sin 2n\alpha \cos 2n\theta \right\}$$

where p is applied pressure, R is the radius of the disc, r and θ are the polar coordinates of a point in the disc, and α is the half central angle related to the distributed load applied. It is seen from the equation that the magnitude of α affects the stress distribution in a disc directly.

Theoretical tensile stresses at the centre of the disc were found to be 9.59 MPa, 6.61 MPa, 8.21 MPa and 8.85 MPa for the standard Brazilian test, and 15°, 20° and 30° loading arcs, respectively (Figure 4.18).

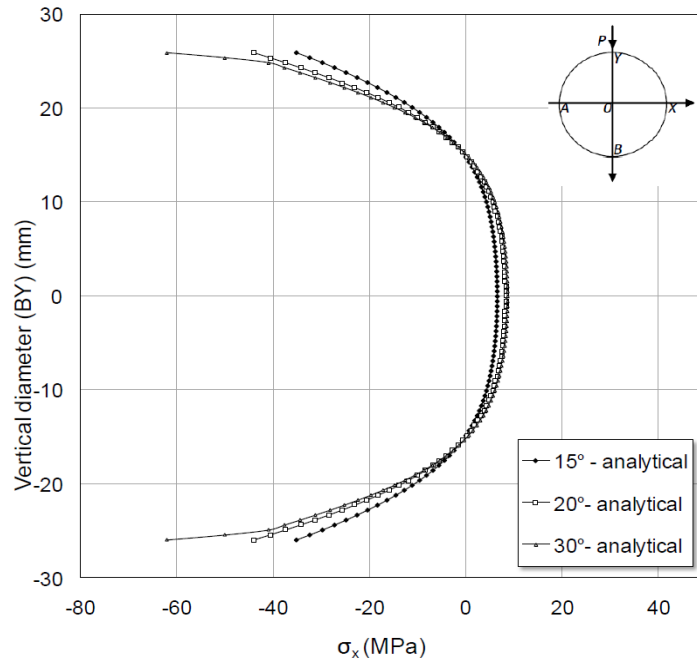


Figure 4.18 Analytical stress distribution along vertical diameter (BY) of the disc (tensile stresses are positive)

In Figure 4.19, the theoretical values of the stresses are compared with the numerical modelling results obtained by FRANC2D to examine the validity of numerical results for modelling in the following sections. In all simulations, the stress distribution results calculated theoretically are quite consistent with the numerical results; however, discrepancies approaching the boundaries are very clear. These may result from the assumed boundary conditions of the two analysis types and/or mesh and geometry specifications of the numerical simulations. As shown in Figure 4.19, tensile stress distribution is maximum at the centre of the disc ($r = 0$) and effective along more than half of the diameter of the specimen. Because the compressive regions took place under the loading

boundaries, compressive stress values of numerical results at the upper half of the disc are bigger than those at the lower part of the disc. In general, analytically calculated tensile stresses are more concave through the vertical diameter than the numerically calculated tensile stresses. However, $2\alpha = 15^\circ$ arc simulation results give the most expanded but smallest value tensile stress distribution along vertical diameter of the disc (Figure 4.19). The details of the simulations are given in Table 4.

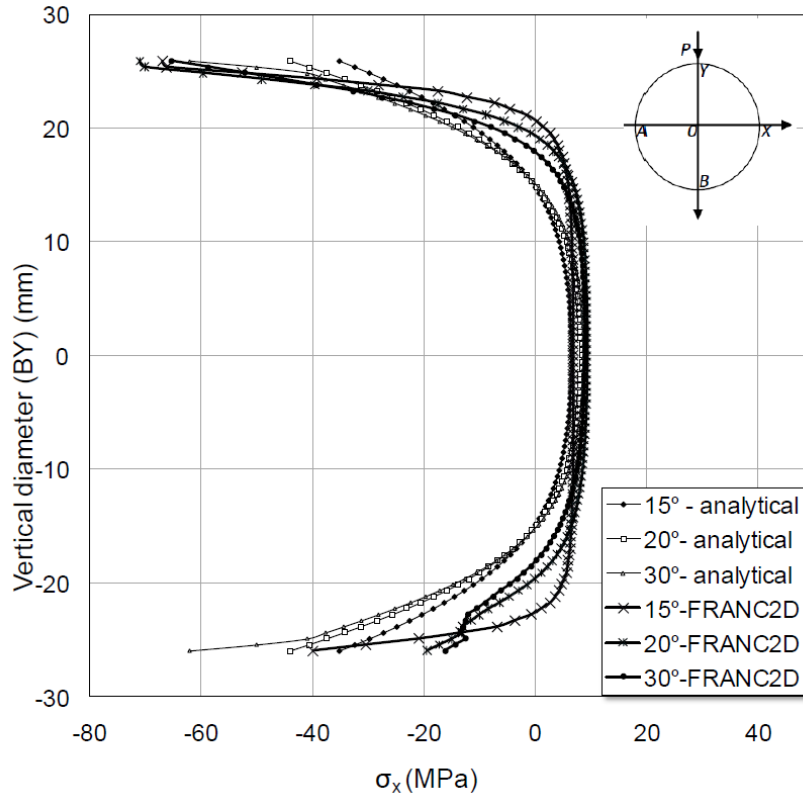


Figure 4.19 Horizontal stress distribution along the vertical diameter (BY) of the disc (tensile stresses are positive)

4.3.4 Comparison between experimental, analytical and numerical results

Recent research (Cai & Kaiser, 2004; Lanaro *et al.*, 2009) has shown that the formula for calculating the indirect tensile strength from failure loads obtained by Brazilian tensile testing might not be applicable for rocks containing minerals with different strengths, flaws and cracks. In previous sections, both numerical and analytical solutions showed that the stress distribution in a disc specimen depends strongly on the loading conditions. However, in both numerical and analytical solutions the rock specimen is assumed to be homogenous, isotropic and linearly elastic. Because pre-existing or newly initiated flaws and cracks affect the stress distribution in the specimens, such specimens cannot be assumed to comply with this assumption. Further, the closed-form analytical solution assumes that failure occurs in pure tension. Some of the available studies

show that failure may occur in both tension and shear (Lanaro *et al.*, 2009; Van de Steen *et al.*, 2005).

The decrease in tensile strength of an actual brittle material is due to the presence of pre-existing cracks in the material (Whittaker *et al.*, 1992). Inglis (1913) and Griffith (1920) were the first researchers to realise the significance of pre-existing cracks as precursors to failure. In this section, vertically aligned 2 mm central pre-existing noncohesive cracks were placed along the vertical diameter of the disc in FRANC2D simulations (Figure 4.20), to assess their effect on the calculated tensile strength. Because crack propagation was not modelled in the simulations, any crack propagation theories FRANC2D uses (e.g., on the basis of maximum tangential tensile stress criterion) were not specifically included in the stress calculations at the tip of the pre-existing cracks. Further, the pre-existing cracks were small compared with those created to test fracture toughness and, hence, the geometry of fracture toughness test was not simulated.

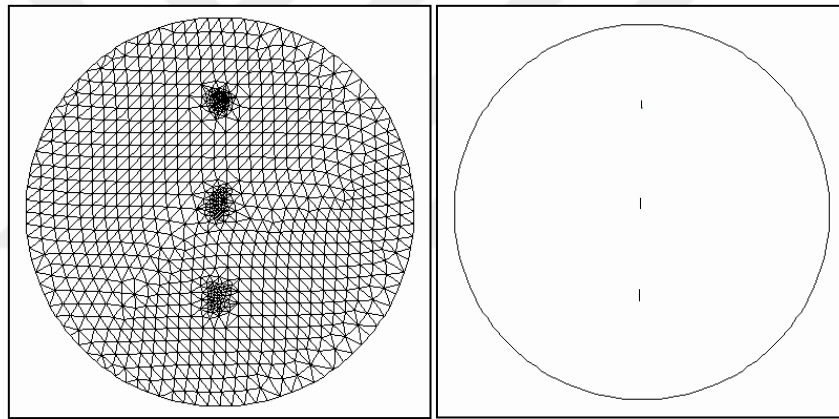


Figure 4.20 Three pre-existing cracks along the vertical diameter of disc

An example to show the effect of pre-existing cracks on stress distribution is given in Figure 4.21. Symmetrical tensile stress concentration zones were produced at the tips of the central pre-existing crack in the $2\alpha = 20^\circ$ loading arc simulations. However, those symmetrical tensile stress zones are arrested and do not extend to the specimen boundaries. Higher value, more concentrated tensile stress zones at the tips of the pre-existing cracks were found in both the $2\alpha = 20^\circ$ and $2\alpha = 30^\circ$ loading arc simulations.

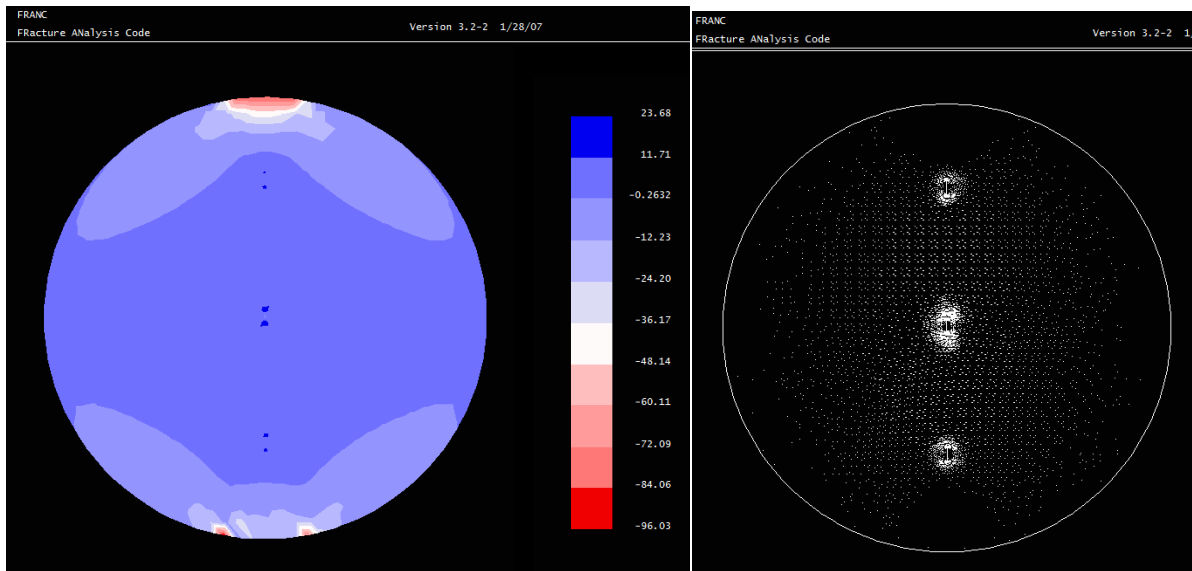


Figure 4.21 (a) Stress distribution in x direction, and (b) Tensile stress bars under 20° loading arc simulation for a disc with three pre-existing cracks (+: tension; -: compression)

The calculated tensile stresses across the horizontal diameter of the disc, across the tips of the pre-existing central cracks, are shown in Figure 4.22. It can be seen that there are much higher tensile stresses due to the presence of a central pre-existing crack, compared to the situation without cracks (Figure 4.16). The highest tensile stress at the central pre-existing crack was obtained for the standard Brazilian jaws.

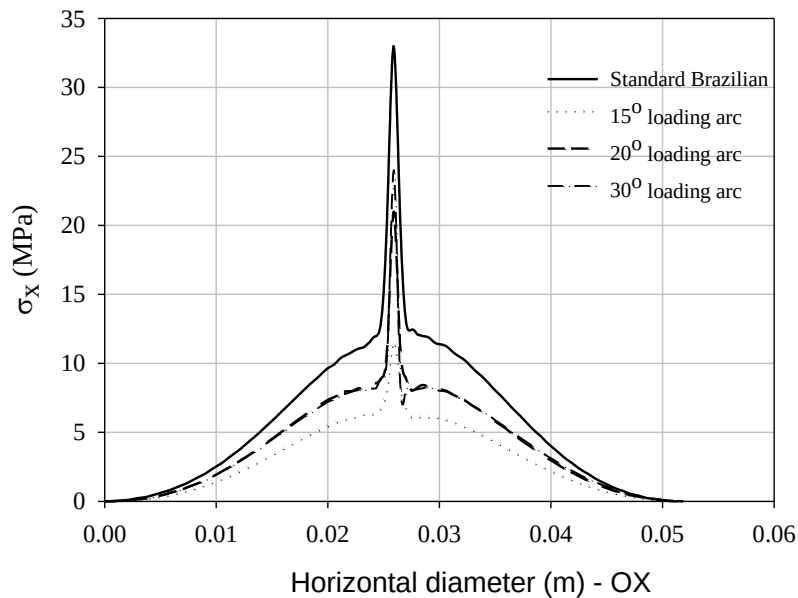


Figure 4.22 Simulated tensile stress distributions across the horizontal diameter of disc through the central pre-existing cracks

To further analyse the effect of heterogeneity on the strength properties of rocks, additional numerical simulations were conducted by RFPA2D (Rock Failure Process Analysis) code with a

heterogeneous material model. The simulated results not only predict the tensile stress in a disc specimen, but also visually reproduce the progressive fracture process. The main advantages of the RFPA model over other models are as follows: (1) by introducing the heterogeneity of rock properties into the model, the RFPA model can simulate nonlinear deformation of a quasi-brittle rock with an ideal brittle constitutive law for the material; (2) by introducing the reduction of material parameters after element failure, the RFPA model can simulate strain-softening and discontinuum mechanics problems in a continuum mechanics mode and (3) by recording the event rate of failed elements, the RFPA model can simulate microseismicities associated with the progressive fracture process. The details of the RFPA2D can be found in Liu's (2004) research.

In this study, a numerical model for the BTS test is built according to the experimental geometry, i.e., the diameter of the disc is 52 mm. Figure 4.23 shows the progressive fracturing of heterogeneous Brazilian disc samples under standard Brazilian jaws and loading arcs. As the loading increases, it is seen that the cracks start at some points close to the upper contact along the vertical diameter of the disc under standard Brazilian jaws (Figure 4.23a). Then cracks unstably propagate radially outward in the upper half of the disc causing a diametral fracture plane with small crack coalescence.

When the angle of the steel arcs increased, it was difficult for fractures to open directly under the loading point as a result of the constraint stress in the horizontal direction under the loading arcs. It is also interesting to mention that any crushed zone just below the loading arcs due to the high compressive loading was not observed in numerical simulations. The first failure/crack initiation occurred at the centre of the disc in all loading arc simulations. In contrast, primary crack initiation occurred very close to the loading point with standard Brazilian simulations, although primary crack initiation took place at the centre of the disc with 15°, 20° and 30° loading arc simulations (Figure 4.23b, c, d). At the end of all fracture process simulations, crack coalescence and failure took place along the vertical diameter of the discs.

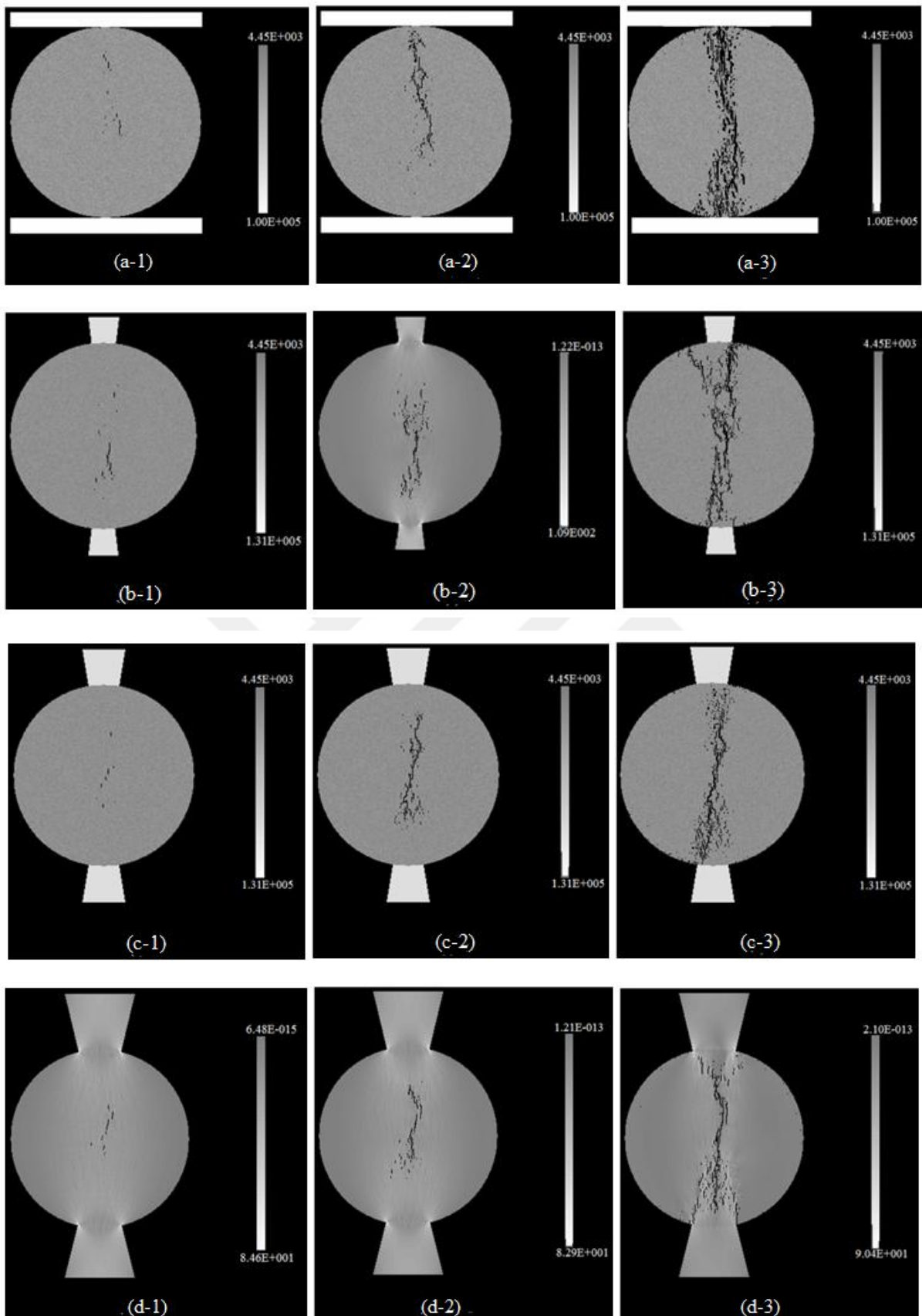


Figure 4.23 Progressive fracture process induced in (a) standard Brazilian jaws (b) 15° loading arcs (c) 20° loading arcs (d) 30° loading arcs

Figure 4.24 and Figure 4.25 show the simulated fringe patterns of the maximum principal stress and minimum principal stress induced in disc specimens subjected to the same displacement loading under different loading geometries. The stress distribution near the centre of disc under diametral loading is quite uniform for both Brazilian jaws and diametral loading arc cases. The stress distribution in the y direction is more uniform than in the x direction. When the angle of the loading arcs increases, the maximum principal stress distribution (compressive stress) becomes more complex along the loading direction compared with standard Brazilian results (Figure 4.24). In contrast, the minimum principal stress (maximum tensile stress) is more focused at the centre of the disc when the angle of the loading arcs increases (Figure 4.25). Thus, the obtained patterns are of great significance for improving our understanding of the crack initiation point in nontransparent rock material under different diametral loading boundaries. These results show that the crack initiation point moves to the centre of the disc with increasing angle of the loading arcs; similar to results of the progressive fracture process.

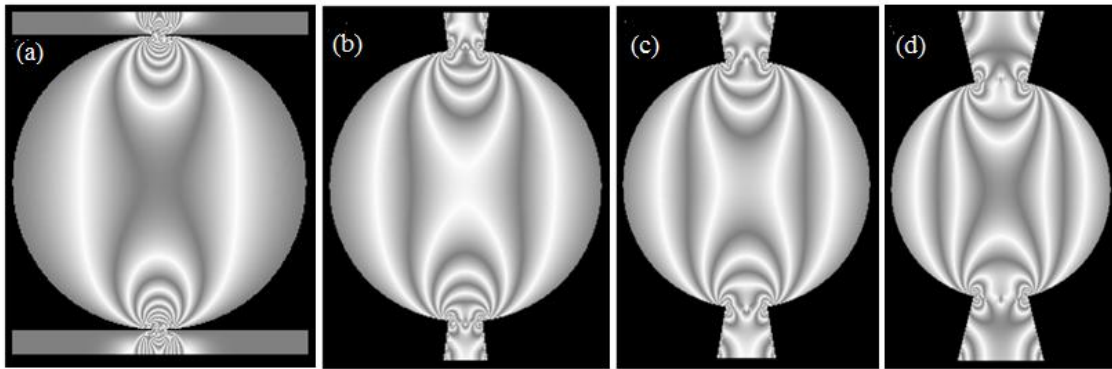


Figure 4.24 Fringe patterns of maximum principal stress induced for (a) standard Brazilian jaws (b) 15° loading arcs (c) 20° loading arcs (d) 30° loading arcs

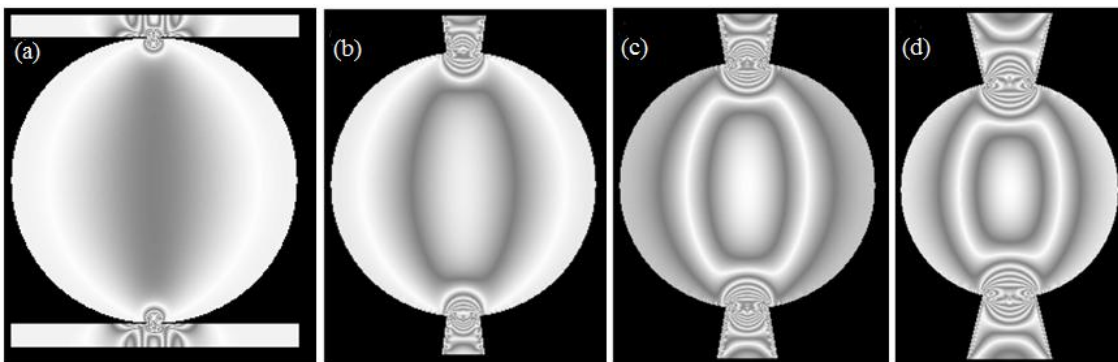


Figure 4.25 Fringe patterns of minimum principal stress induced for (a) standard Brazilian jaws (b) 15° loading arcs (c) 20° loading arcs (d) 30° loading arcs

All experimental, numerical and analytical results show that the formula for calculating the indirect tensile strength from the failure load obtained by the standard Brazilian test might not represent the actual tensile strength of brittle rocks. This is because the contact area between the standard Brazilian loading jaws and the rock disc specimen cannot be considered as a line. In the experiments described herein, the disc specimens shattered into several pieces under the standard Brazilian jaws. In contrast, applying distributed diametral loads with loading arcs causes single failure planes along the loaded diameter in all three rock types tested.

The indirect tensile strength of Brisbane tuff was calculated as 9.2 MPa using the formula proposed by ISRM (Table 4.4). According to the analytical solutions of Hondros (1959), when $2\alpha < 11^\circ$ and r/R tends to zero, the tangential stress along the vertical diameter may be calculated using the same formula. This indicates that the formula proposed by ISRM is not just for line loading but also for diametral loading over an arc of up to $2\alpha = 11^\circ$. This outcome was verified by Jaeger and Cook (1976). They found from experiments with distributed loads over a narrow arc, usually up to about $2\alpha = 15^\circ$, that the tensile strength varied little from that obtained using a line load and gave similar diametral extension fractures. In contrast, the tensile strength decreases to 6.61 MPa, when $2\alpha = 15^\circ$ is used in theoretical solutions (Table 4.4). This result raises another doubt about the applicability of the formula proposed by ISRM to determine the indirect tensile strength of rocks.

Although the highest failure load was found experimentally and in both numerical models with FRANC2D and RFPA2D with the 30° loading arc (except the Brazilian line loading simulation), the highest tensile stress was found with Brazilian tests after analytical calculations (Table 4.4). This result shows that further theoretical studies are needed for formulation of line and diametral loading on disc specimens.

Table 4.4 Comparison of experimental, numerical and analytical results for Brisbane tuff

Loading mode	Experimental ultimate load (kN)	Projected width of loaded section of disc (mm)	Theoretical tensile stress at centre of disc (MPa)	Numerical tensile stress at centre of disc (MPa) (FRANC2D)	Numerical tensile stress at centre of disc (MPa) (RFPA2D)
Standard Brazilian jaws	19.60	1.00	9.20	12.36 *	7.91
15° loading arc	14.81	6.80	6.61	6.88	8.04
20° loading arc	19.20	9.07	8.21	8.79	8.29
30° loading arc	22.30	13.60	8.85	8.98	8.70

* Calculated assuming $\alpha = 1^\circ$

Although good agreement was found between experimental and numerical simulation results in terms of fracture progress, it is not easy to decide the correct tensile stress value representing indirect tensile strength of rocks. Thus, some empirical relationships between intrinsic material properties were used to clarify this. One of the well-known empirical relations between mode I fracture toughness, K_{IC} , and the tensile strength of rocks, σ_t , was given by Zang (2002). The mode I fracture toughness of Brisbane tuff was found experimentally as $1.18 \text{ MPa}\sqrt{\text{m}}$, using one of the ISRM suggested methods, the Cracked Chevron Notched Brazilian Disc (CCNBD; Fowell, 1995). CCNBD specimens have the same diameter and thickness as standard Brazilian disc specimens. Thus, adding fracture toughness into the investigation of the indirect tensile strength of rocks is quite meaningful. The relationship between fracture toughness and tensile strength is given by Zang (2002):

$$K_{IC}(\text{MPa}\sqrt{\text{m}}) = 0.145\sigma_t(\text{MPa}) \quad (4.4)$$

According to Eq. (4.4), the expected indirect tensile strength is 8.13 MPa when the experimentally obtained mode I fracture toughness value $1.18 \text{ MPa}\sqrt{\text{m}}$ is used. The maximum stress intensity value closest to the experimental fracture toughness of $1.18 \text{ MPa}\sqrt{\text{m}}$, was obtained with $2\alpha = 20^\circ$ loading arc simulations ($1.21 \text{ MPa}\sqrt{\text{m}}$) by using FRANC2D. In order to support that result, another well-known empirical relation between the mode I fracture toughness, K_{IC} , and the tensile strength of rocks, σ_t , is given by Whittaker *et al.* (1992). Whittaker *et al.* (1996) found that fracture toughness is dependent on other physico-mechanical parameters of rocks, such as their tensile strength,

compressive strength, and Poisson's ratio. The empirical relationship between mode I fracture toughness and the tensile strength of brittle rocks is given by (Whittaker, 1996):

$$K_{IC}(\text{MPa}\sqrt{\text{m}}) = 0.27 + 0.107\sigma_t(\text{MPa}) \quad (4.5)$$

According to Eq. (4.5), the expected indirect tensile strength is 8.5 MPa when the experimentally obtained mode I fracture toughness value $1.18 \text{ MPa}\sqrt{\text{m}}$ is used. The maximum stress intensity value closest to the experimental fracture toughness of $1.18 \text{ MPa}\sqrt{\text{m}}$, was obtained with $2\alpha = 20^\circ$ loading arc simulations ($1.21 \text{ MPa}\sqrt{\text{m}}$) by using FRANC2D (8.79 MPa, Table 4.4).

In contrast, the given indirect tensile stress values obtained by RFPA2D are very close to one another for various loading geometries. Hence, the above empirical relationships will not work for the very close stress values obtained by RFPA2D.

4.4 Chapter Summary

From indirect tensile tests on Brisbane tuff specimens, it was concluded that both tensile fracturing and indirect tensile strength of rocks are strongly dependent on the type of diametric loads. A general increase in failure load was obtained with increasing angle of loading arc for all three rock types tested. The maximum recorded failure load, corresponding to the finest and most centrally located crack, was obtained with a $2\alpha = 30^\circ$ loading arc. The highest standard deviation of the failure loads was obtained using the standard Brazilian jaws, which resulted in catastrophic specimen failure. In contrast, the direct tensile strength of Brisbane tuff was found to be lower than the indirect tensile strength, as a result of the difficulty of eliminating torsional effects in the direct tensile test.

Numerical simulations indicated that the maximum tensile stress appears at the centre of the disc under the standard Brazilian jaws, while the lowest tensile stress appears at the centre of the disc under a $2\alpha = 15^\circ$ loading arc. Although the highest failure load was found experimentally and in both numerical models using FRANC2D and RFPA2D with the 30° loading arc (except the Brazilian line loading simulation), the highest tensile stress at the centre of the disc was found with standard Brazilian tests after analytical calculations. Moreover, it was found that while the theoretical horizontal stress distributions along the vertical diameter are reasonably consistent with the numerical results, the analytically calculated tensile stresses are less uniform.

Numerical simulations incorporating pre-existing cracks showed much higher and more concentrated tensile stresses at the tips of these cracks. Thus, rock heterogeneity due to pre-existing cracks and defects are found to be important for determination of indirect tensile strength of rocks.



CHAPTER 5

DETERMINATION OF FRACTURE TOUGHNESS OF ROCKS— METHODS AND RESULTS

Rock fracture mechanics has been widely used for mechanical fragmentation, crushing and tunnel boring, rock slope analysis, explosive fracturing, hydraulic fracturing, geophysics, earthquake mechanics, geothermal energy extraction and many other practical problems. Rock fracture toughness is a key parameter in rock fracture mechanics used to forecast crack initiation and propagation, which is important in the design of rock boring equipment, rock bursting, forecasting of rock drilling forces, wellbore stability and stability of jointed rock masses. As a result, it is important to test rock fracture toughness with adequate accuracy. To assess fracture toughness values of brittle materials like rocks, the ISRM suggests three methods: SR specimen (Ouchterlony, 1988), CB specimen (Ouchterlony, 1988) and CCNBD specimen (Fowell *et al.*, 1996). In this study, the two following testing methods were performed to compare results and yield consistent fracture toughness values: CCNBD and SR tests.

5.1 Methods

Comparisons between ISRM suggested methods have been mostly focused on SR and CB specimens (Cui *et al.*, 2009), or on CCNBD and CB specimens (Iqbal & Mohanty, 2006). Although some consistency has been provided by these suggested methods, a variation of 20–30% was still obtained for the same rock type. Recent research has indicated that CCNBD values are markedly smaller than other comparable tests (Cui *et al.*, 2009; Wang *et al.*, 2004). However, the consistency of tested mode I fracture toughness between CCNBD and SR specimens has not been validated adequately and needs to be compared after performing more experiments. In this research, SR and CCNBD tests were conducted on Brisbane tuff specimens and the results were compared to find the most accurate mode I fracture toughness value of Brisbane tuff.

5.1.1 Cracked Chevron Notched Brazilian Disc test

The geometry of a CCNBD specimen is illustrated in Figure 5.1. The chevron notch causes crack propagation to start at the tip of the V alignment and to proceed radially outwards in a stable fashion until the point where the fracture toughness is calculated.

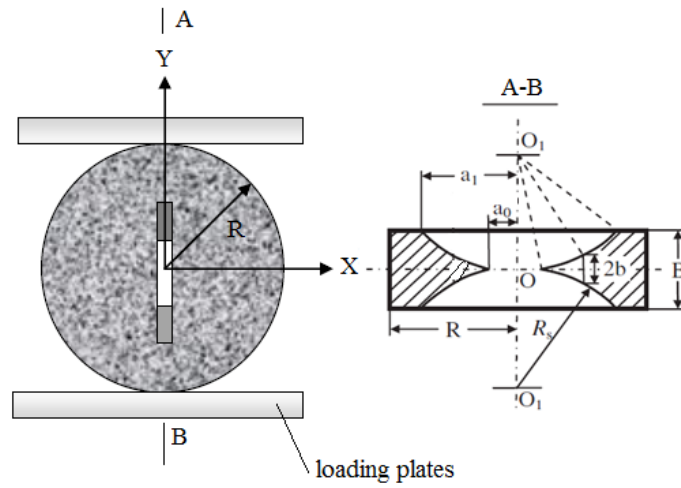


Figure 5.1 The CCNBD specimen geometry with recommended test fixture

All dimensions of the geometry should be converted into dimensionless parameters with respect to the specimen radius, R , and diameter, D . The suggested standard specimen dimensions are given in the ISRM suggested methods (ISRM, 1995). Other selections of specimen geometrical dimensions are possible, but in order to have a valid test, the two most important selected dimensions; dimensionless final notched crack length (α_1) and the dimensionless quantity, α_B , must fall within the range outlined in Figure 5.2 (ISRM, 1995). The dimensionless initial crack length (α_0 ; a_0/R), dimensionless final notched crack length ($\alpha_1 = a_1/R$) and α_B (B/R) are the three basic dimensions for the CCNBD parameters. All specimen geometries used in this research were in the valid ranges indicated by the red circle in Figure 5.2; the full validity range is also shown.

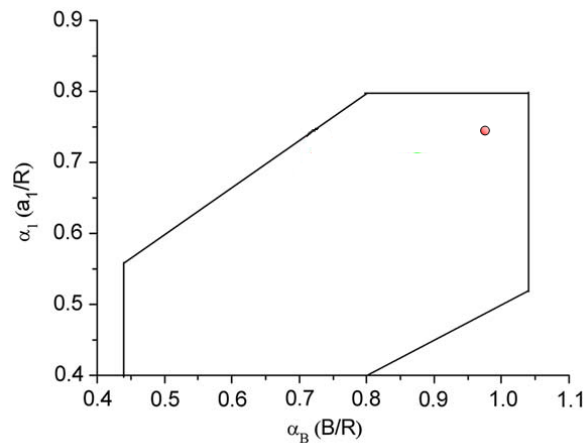


Figure 5.2 Valid geometrical ranges of CCNBD specimens for fracture toughness testing

For fracture toughness testing, the important stress intensity factor value for the specimen is the minimum stress intensity factor, denoted as Y_{min}^* , as it corresponds to the failure load (P_{max}) recorded during testing. The fracture toughness value of a CCNBD specimen is calculated as follows:

$$K_C = \frac{P_{max}}{B\sqrt{R}} Y_{min}^* \quad 5.1$$

where P_{max} is the maximum failure load in N, B is the thickness in mm, R is the radius of the CCNBD specimen in mm, and Y_{min}^* is the critical dimensionless stress intensity factor. For CCNBD specimens within the valid range shown in Figure 5.2, the critical dimensionless stress intensity value (Y_{min}^*), which is determined by the specimen geometry dimensions α_0 (a_0), α_1 (a_1) and α_B (B) only, can be calculated as given by the ISRM (1995):

$$Y_{min}^* = u \cdot e^{v \cdot \alpha_1} \quad 5.2$$

where constants u and v are listed in the ISRM suggested methods (ISRM, 1995).

Most of the tests were carried out on Brisbane tuff, because it is a host rock of Brisbane's first motorway tunnel, CLEM7, from which core samples were obtained. A circular 40 mm diamond saw was used to cut the required notch (Figure 5.3a). A specially designed jig recommended by ISRM was used to ensure that the chevron notches are exactly in the centre of the disc. Additionally, the jig was controlled by an electronic alignment device with displacement calculated on x, y, z coordinates for accuracy in controlling the centre alignment and penetration of the saw (Figure 5.3b).

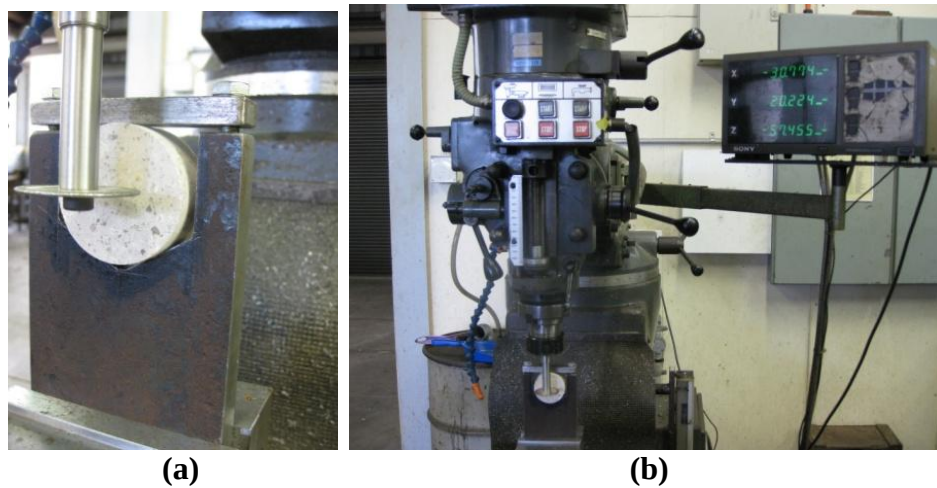


Figure 5.3 CCNBD specimen preparation equipment

The thickness, t , of the notches was 1.4 mm and the thickness of the sample, B , was 25 mm. The initial chevron notch crack length, a_0 , was 7.5–8 mm and the final chevron notch crack length was

36–37 mm. Two kinds of Brisbane tuff CCNBD specimens and loading platens are shown in Figures 5.4a and 5.4b. An Instron 6027 with 100 kN load cell hydraulic servo-control testing system with Bluehill software was used to conduct the tests. Diametral displacement parallel to the loading direction was measured by means of an extensometer with 50 mm gauge length. The crack displacement was measured as CMOD across the crack mouth. A clip gauge for measuring the notch opening was attached to the knives. An Instron 2670 series crack opening displacement gauge was used to measure CMOD (Figure 5.4c). The gauge length was 10 mm and maximum travel was 2 mm. The gauge met the requirements set out in British standard BS 5447 and American standard ASTM 399 70T.

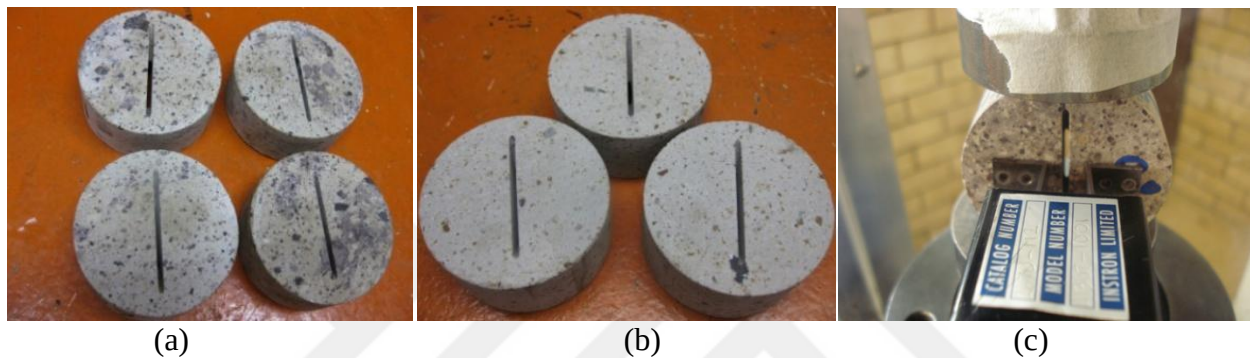


Figure 5.4 (a), (b) Prepared CCNBD samples and (c) loading plates and CMOD transducer

Six CCNBD samples were tested under load control test conditions. The specimens were placed under the platens with zero crack inclination angle ($\theta = 0^\circ$) to provide mode I loading conditions. The loading rate was chosen as 9 kN/s to cause failure within 20 s as suggested by ISRM (1995). The load and CMOD readings were recorded continuously until failure. Results are given in following sections.

5.1.2 Short Rod test

This method is used to evaluate mode I fracture toughness values only. The SR method was introduced by Barker (1977) and it became an ISRM-suggested method after the studies by Ouchterlony (1987, 1988). The specimen is a typical core-based one with a machined notch along the rod axis. The external load is applied to the notch mouth to split the specimen into two identical halves. Because tensile load is directly applied perpendicular to the initial notch plane, this may result in bonding failures at the specimen-loading platen contacts, especially for hard rock types. Figure 5.4 shows schematically the specimen configuration and the loading and dimension notations. The specimen geometry is restricted in dimensions as listed in Figure 5.5.

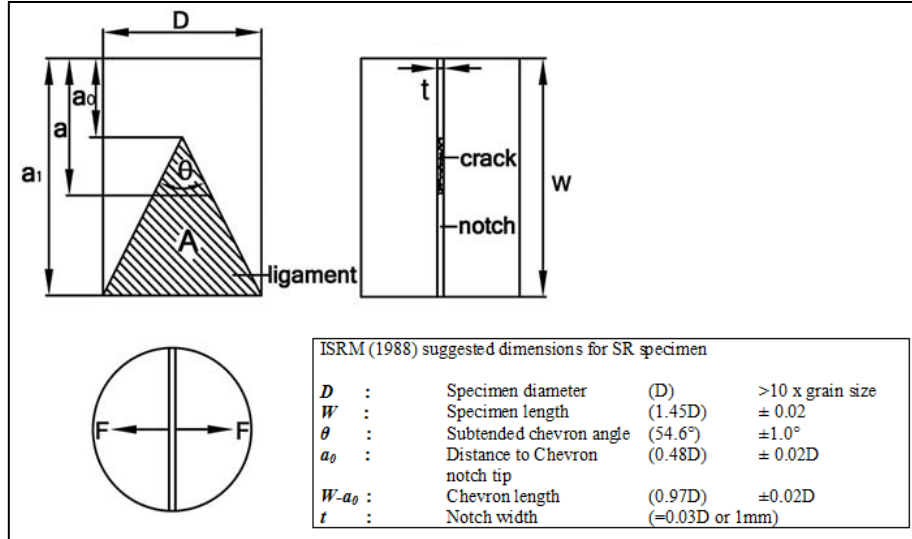


Figure 5.5 Chevron notched Short Rod specimen and associated notations

The SR test has two levels. Level I testing requires only recording of the peak load, which is used to evaluate the level I fracture toughness. This test should be performed under load control with an average stress intensity factor rate K_{SR} of more than or equal to $0.25 \text{ MPa}\sqrt{\text{m/s}}$. Level II testing requires the recording of the complete load versus the CMOD (Crack Mouth Opening Displacement) curve. The level II fracture toughness, which accounts for the effect of nonlinearity, is determined depending on whether the two unloading lines span the evaluation point (ISRM, 1988). This test should be performed in displacement control with a CMOD rate faster than $0.017 K_{SR}/(E\sqrt{D}) \text{ m/s}$. K_{SR} is the level I fracture toughness, E is Young's modulus and D is the diameter of the specimen.

The fracture toughness calculation is achieved by considering the following equation, which was proposed by ISRM (1988) for Level I testing:

$$K_{SR} = C_K 24.0 P_{\max} / D^{1.5} \quad 5.3$$

where K_{SR} is fracture toughness ($\text{MPa}\sqrt{\text{m}}$), $P_{\max}$ is failure load (MN), D is the specimen diameter (m), and C_K is the correction factor to account for the size variation of the specimen, given by

$$C_K = 1 - \frac{0.6\Delta W}{D} + \frac{1.4\Delta\alpha_0}{D} - 0.01\Delta\theta \quad 5.4$$

where ΔW is variation in specimen height W (m), Δa_0 is the initial position of the chevron notch apex (m) and $\Delta\theta$ is the chevron notch angle (in radians).

The Brisbane tuff specimens' diameter and length were 52 mm and 75 mm respectively. The chevron angle, θ , was 55° , and the chevron tip position was at 25 mm. Figure 5.6 shows the SR samples before testing. The end plates were mounted parallel to and equidistant from the notch centre plane.



Figure 5.6 Brisbane tuff Short Rod specimens before testing

Each specimen was placed in the testing machine in such a way that the load transfer system was properly aligned. A small bias load was used to secure it in the correct position. Level I presumes that the test will be run under load control. The load was recorded and the test was run until the specimen failed. The average stress intensity rate during the test was $0.25 \text{ MPa}\sqrt{\text{m}}/\text{sec}$ so that failure occurs within 10 s of initial load application. Four SR samples were tested and results are given in the following sections.

5.2 Results

In general, a load to achieve opening mode crack propagation can be applied to the specimen by direct tension, compression or bending. These two SR and CCNBD methods could be classified in terms of their loading conditions. The SR method is one of the methods suggested by ISRM (1981) in which a tensile load is directly applied perpendicular to the initial notch plane. In contrast, a CCNBD specimen is loaded in compression along the diametral plane. Hence, comparison of these two methods has attracted attention in recent research. In this study, numerical and analytical methods were both included to calculate more accurate fracture toughness values for the Brisbane tuff.

5.2.1 K_{IC} determined by the CCNBD and SR methods for several Brisbane tuff types

Failure load and fracture toughness values of the Brisbane tuff determined with the CCNBD method were compared to the results of SR tests. In general it was observed that the maximum failure loads for CCNBD specimens are higher than those for SR specimens. Figure 5.7 shows the SR and CCNBD samples after testing.

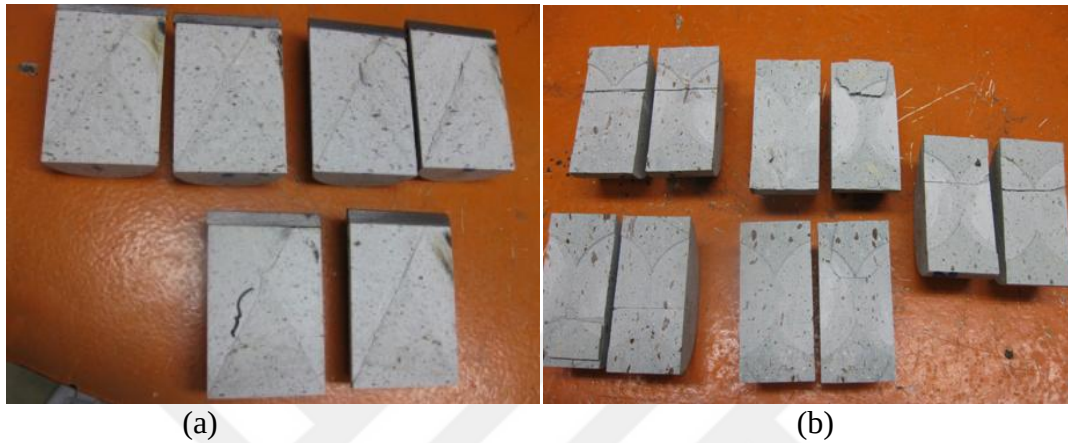


Figure 5.7 (a) SR and (b) CCNBD specimens after testing

The maximum recorded load and the calculated fracture toughness values obtained from both ISRM standard tests are shown in Table 5.1.

Table 5.1. Mode I fracture toughness values of Brisbane tuff obtained by SR and CCNBD tests

Rock type	P_{max} (kN)	K_{IC} (SR)	P_{max} (kN)	K_{IC} (CCNBD)
	SR	(MPa $\sqrt{m}$)	CCNBD	(MPa $\sqrt{m}$)
Brisbane tuff-1	1.0	2.02	5.25	1.38
Brisbane tuff-1	0.99	2.01	5.1	1.34
Brisbane tuff-1	1.3	2.63	6.18	1.63
Brisbane tuff-1	0.85	1.72	5.87	1.55
Average	1.03	2.1	5.6	1.48
Brisbane tuff-2	1.10	2.23	4.68	1.21
Brisbane tuff-2	1.07	2.18	6.47	1.68
Brisbane tuff-2	1.12	2.26	6.35	1.64
Brisbane tuff-2	1.05	2.12	7.10	1.83
Average	1.08	2.19	6.15	1.59

As seen in Table 5.1, mode I fracture toughness values of Brisbane tuff obtained for the CCNBD specimens are lower than those obtained by the SR method although bigger failure loads were obtained with CCNBD tests than with SR tests.

According to the rock fracture mechanics, the stress intensity factor (K_I) decreases with increasing crack length under a constant applied load and then a sharp crack initiates at the tip of the chevron notch, requiring an increasing load to grow stably at $K_I = K_{IC}$ to the notch base. After that propagated crack, unstable crack propagation starts because further growth would not require an increasing load. Thus, fracture toughness (K_{IC}) is calculated as that recorded ultimate load and the start of unstable crack propagation. According to the Fowell and Xu (1996), the critical crack length a is between the initial chevron notch crack length, a_0 , and the final chevron notch crack length, a_1 .

An analysis was done to obtain critical crack length in front of the chevron tip using the Straight-Through Crack Assumption (STCA) approach (Shetty *et al.*, 1985). Figure 5.8 shows the normalised stress intensity factor versus normalised crack length for both CSTBD and CCNBD geometries. Wang *et al.* (2003) assumed that the critical crack length is between the initial chevron crack length and final chevron notch crack length. According to Fowell and Xu (1996), the critical crack length a is between the initial chevron notch crack length, a_0 , and the final chevron notch crack length, a_1 . As shown in Figure 5.8, unstable crack propagation starts at the dimensionless crack length where the dimensionless stress intensity factor is a minimum (Y_{min}^*) which equals to 1.21, and the relevant fracture toughness value of the Brisbane tuff is $1.74 \text{ MPa}\sqrt{\text{m}}$ where the critical crack length is equal to 13 mm ($\alpha = 0.5$). This critical crack length indicates that unstable crack propagation starts between the initial chevron notch crack and final chevron notch crack lengths.

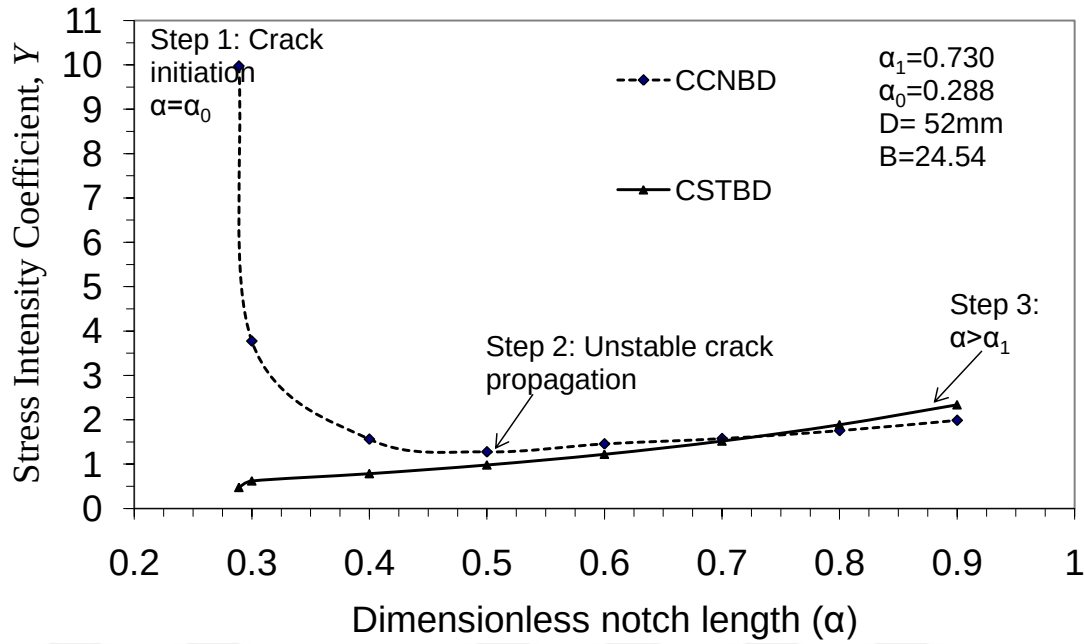


Figure 5.8 Dimensionless stress intensity factor for CSTBD and CCNBD specimens

It should be pointed out that the value of $Y_{\min}^* = 0.87$ obtained using the formula given by ISRM (1995) is too small when it is compared with the corresponding value of $Y_{\min}^* = 1.21$ obtained by STCA method. The obtained fracture toughness values are different because of different dimensionless stress intensity factor values. A detailed discussion of these differences is given in the ‘Discussion’ section of this chapter.

5.2.2 K_{IIC} and mixed-mode fracturing of rocks determined by the CCNBD method

Fractures within a rock mass can be generated both in tensile and compressive stress fields and are therefore very common. The details are given in the relevant research literature in Chapter 2. Very little work has been conducted on mode II fracture toughness testing, let alone development of standard testing methods. At present there is no ISRM-suggested method for determination of mode II rock fracture toughness.

Three rock types, granite, Brisbane tuff and sandstone, representing high-, medium- and low-strength categories of rock, respectively, were tested in this study. The granite blocks with 210 MPa UCS were from Brisbane and from the quarry at Keperra. The sandstone samples with 37 MPa UCS were from Helidon, Brisbane. All samples were fairly isotropic. The thickness, t , of the notches was 1.5 mm and the thickness of the samples, B , was 25 mm. The initial chevron notch crack length, $2a_0$, was 16–18 mm and the final chevron notch crack length, $2a_1$, was 36–37 mm.

The three kinds of CCNBD specimens, loading platens and CMOD transducer are shown in Figure 5.9. An Instron 6027 with 100 kN load cell hydraulic servo-control testing system with Bluehill software was used to conduct the tests. The crack displacement was measured as CMOD across the crack mouth. An Instron 2670 series crack opening displacement gauge was used to measure CMOD (Fig. 5.7c). The gauge length was 10 mm and maximum travel was 2 mm. The gauge met the requirements set out in British standard BS 5447 and American standard ASTM 399 70T.

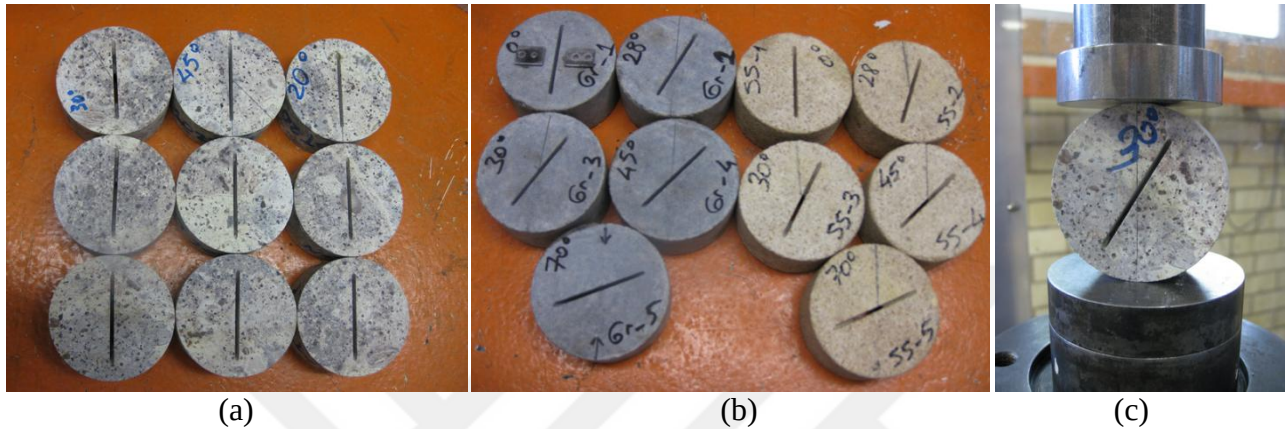


Figure 5.9 Prepared CCNBD samples (a) Tuff (b) Granite and sandstone (c) Loading platens

Disk specimens were diametrically loaded with various angles (β) inclined according to diametral compressive loading directions ranging from 0° to 70° . A load-controlled testing manner was adopted and loading was continued till failure. Load, diametral displacement and crack mouth displacement were continually recorded during the test using a computerised data logger.

In order to investigate the effect of crack inclination on crack initiation and propagation direction, three different strength rock specimens were tested. Of course, there are various fracture patterns under different planar loading on the original crack plane. Our loading type is diametral compressive loading to create compression-shear loading on the pre-existing crack plane. Because the crack initiation angle is positive (counter clockwise), the crack propagation direction tends to be parallel to the direction of compressive loading in all three rock types (Figure 5.10). In general, the ultimate failure load increases up to $\beta = 33^\circ$ crack inclination angle with all rock types (Table 5.2). The reason for that failure load increase may come from the fact that the highest shear stress occurs with this crack inclination angle under this loading geometry. For example, the opposite compressive stress may serve as a shear stress on the original crack plane most effectively.

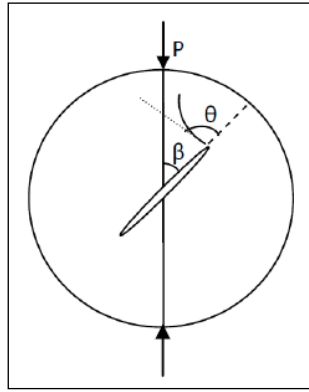


Figure 5.10 CCNBD sample under diametral compression

Table 5.2 Failure loads obtained for Brisbane tuff, granite and sandstone CCNBD samples

	Inclination angle	Failure load (kN)		
	(β)	Brisbane tuff	Granite	Sandstone
Repeat 1	0°	4.4	9.6	1.9
Repeat 2		4.1	9.0	1.7
Repeat 1	28°	4.9	10.2	2.4
Repeat 2		4.7	9.5	1.8
Repeat 1	30°	4.4	10.1	2.1
Repeat 2		4.8	9.6	2.3
Repeat 1	33°	5.3	11.0	2.2
Repeat 2		5.7	11.3	2.5
Repeat 1	45°	4.9	10.2	2.1
Repeat 2		4.4	9.7	2.3
Repeat 1	70°	4.7	9.8	2.5
Repeat 2		4.3	9.4	2.2

More commonly, if a CCNBD specimen is loaded with the crack oriented at an appropriate angle (β) with respect to the loading direction, a mixed-mode I–II stress field can be achieved at the crack tips and the crack propagates at an angle (θ) with respect to its crack plane.

The crack initiation angle (θ) was found to be strongly dependent on the crack inclination angle (β) in our experimental studies. It was found that cracks initiated at an angle of 70–100° relative to the original crack plane when the inclined crack was subjected to uniaxial compressive stress. The tested Brisbane tuff, granite and sandstone samples are shown in Figure 5.11. From the experimental results, cracks initiated at the tip of the notch for all three kind of rock specimens with

values of β up to 30° only. However, one of the most important observations from the experimental findings was that the location of crack initiation moved to the centre of the notch crack with increasing crack inclination angle after 30° ($\beta \geq 30^\circ$). Also, it was found with 70° inclined cracks in sandstone and granite samples that single coplanar shear cracks took place individually at one end of the tip of the cracks.

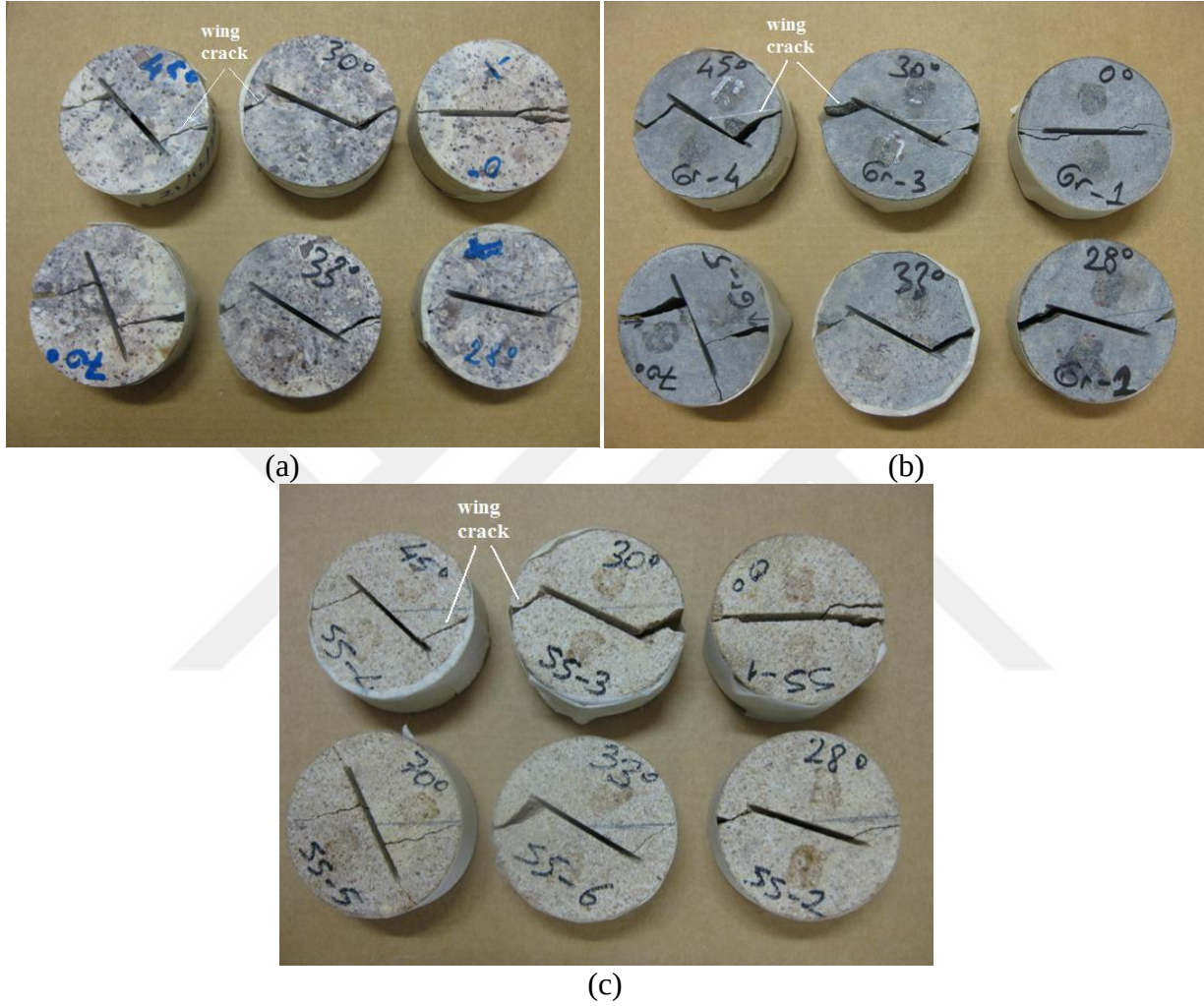


Figure 5.11 Tested CCNBD specimens (a) Brisbane tuff (b) granite and (c) sandstone

Crack initiation angles (θ) depending on the crack inclination angles (β) are shown in Table 5.3 for Brisbane tuff, granite and sandstone samples. In general, crack initiation angle increases with increasing crack inclination angle in all three rock types. From the above discussions and results, it is clear that a diametral compressive loading along the crack plane in CCNBD samples always creates mixed-mode fracturing without any clear mode II fracturing, although the crack is subjected to the compressive-shear loading.

Table 5.3 β and θ values for Brisbane tuff, granite and sandstone samples

Crack inclination angle (β)	Crack initiation angle (θ)		
	Brisbane tuff	Granite	Sandstone
0°	0°	0°	0°
28°	65°	60°	65°
30°	75°	70°	75°
33°	80°	70°	80°
45°	85°	80°	80°
70°	90°	90°	90°

According to the maximum tangential stress criterion, for pure shear loading ($K_I = 0$) the crack initiation angle (θ) should be 70.53°. The experimentally observed crack initiation angle was found to be close to the theoretical angle at the tip of both the $\beta = 30^\circ$ and $\beta = 33^\circ$ inclined notch cracks with the highest compressive loading values. However, it is difficult to assume that the mode I stress intensity factor is zero at that angle. Thus, the applicability of the maximum tangential stress criterion for stress intensity factor evaluations has been criticised in recent research (Al-Shayea, 2005; Rao *et al.*, 2003).

Crack displacement was monitored continuously until failure. Figures 5.12, 5.13 and 5.14 show the crack displacement responses of pre-existing notch crack mouths, with various crack inclination angles (β) for Brisbane tuff, granite and sandstone CCNBD samples, respectively. In general, it is clear that all the notched cracks opened for crack inclination angles (β) of up to 28° for the three rock samples. It can be concluded that effective normal stress perpendicular to the crack plane is mostly tensile, which causes the crack to open. However, notched cracks in the three rock types showed different responses to diametric uniaxial compressive stress with $\beta = 30^\circ$ and $\beta = 33^\circ$ inclination angles. For instance, normal stress perpendicular to the crack plane causes closing of the crack mouth with $\beta = 30^\circ$ and $\beta = 33^\circ$ inclination angles in Brisbane tuff CCNBD samples (Figure 5.12). Then opening starts again, with plastic behaviour up to failure. In contrast, normal stress perpendicular to the crack plane causes crack opening with $\beta = 30^\circ$ inclination angle and crack closing with $\beta = 33^\circ$ inclination angle in granite CCNBD samples (Figure 5.13). Quite different results from the Brisbane tuff and granite samples were obtained with the sandstone samples. The normal stress perpendicular to the crack plane clearly causes crack opening with $\beta = 30^\circ$, however, slight crack closing starts with $\beta = 33^\circ$ inclination angle in sandstone samples (Figure 5.14).

The smallest crack displacement was obtained with granite samples because granite seems the most brittle of the three rock types (Figure 5.13). Another important observation obtained from the load–CMOD graphs is the presence of plastic deformation regions before the peak load points. Because clear plastic deformation took place before failure in all three rock samples, there is a possible Fracture Process Zone (FPZ) in front of the crack tip. The formation of the plastic process zone at the tip of the crack is seen clearly because of the plastic deformation region on the graphs before failure. It is interesting to note that this possible FPZ zone takes place in front of the crack tip propagating in only opening fracturing mode. When the crack closing starts, hardly any plastic deformation takes place before the peak load. Especially, there is no plastic deformation taking place in front of the $\beta = 45^\circ$ inclined crack tip with all rock types. This may show that the fracturing mode is still opening even with crack closing and that effective tensile stress tends to extend to the centre of the crack instead of being suppressed in front of the crack tip.

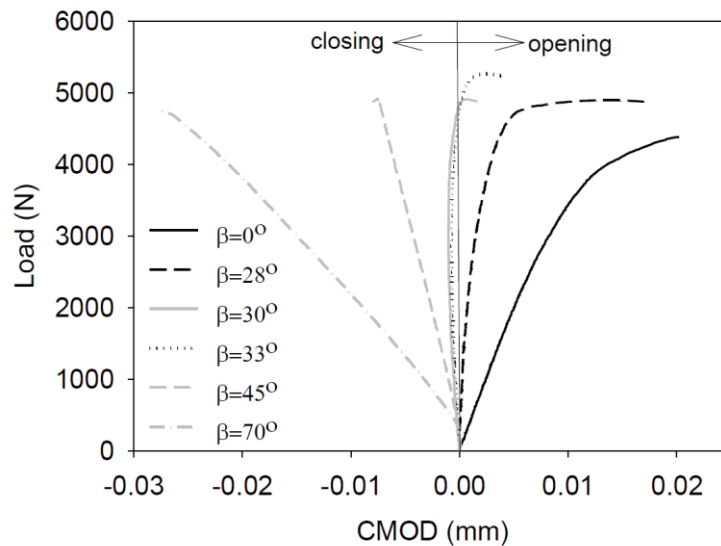


Figure 5.12 Crack displacement under mixed-mode loading of Brisbane tuff CCNBD samples

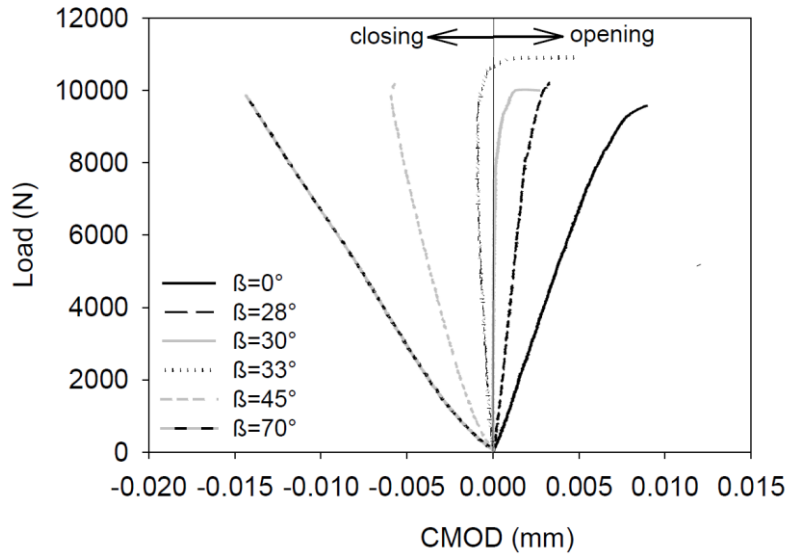


Figure 5.13 Crack displacement under mixed-mode loading of granite CCNBD samples

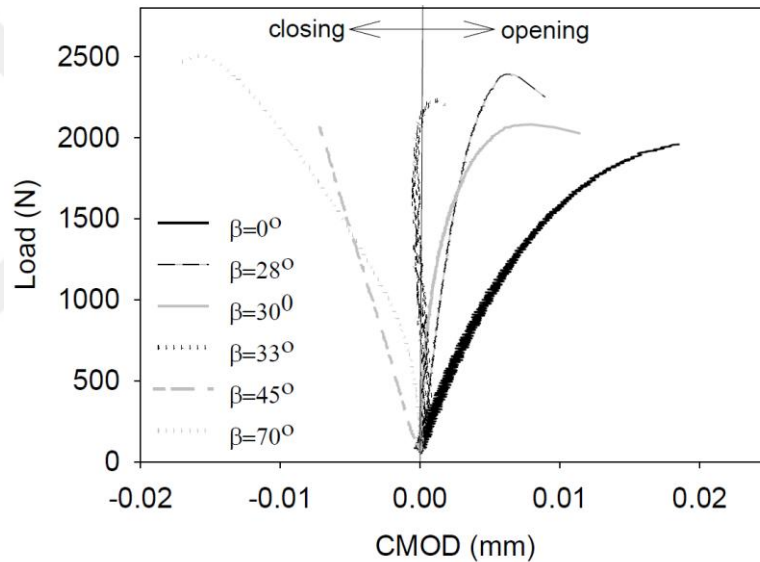


Figure 5.14 Crack displacement under mixed-mode loading of sandstone CCNBD samples

The plots of maximum crack displacements against the crack inclination angles are shown in Figures 5.15, 5.16 and 5.17 for Brisbane tuff, granite and sandstone samples, respectively. The maximum crack closure was observed at a crack inclination of 70° for all three rock samples. The amount of this closure is around 0.02 mm, which is much less than the width of the notch (2 mm). This means that the notch crack surface planes do not come in contact with each other before failure. In contrast, the maximum crack opening was observed at a crack inclination of 0° for all three rock samples. Finally, the smallest crack displacement was obtained with the granite samples at all crack inclination angles (Figure 5.16). This may result from the fact that granite sustained the highest load with the highest stiffness of all three rock types.

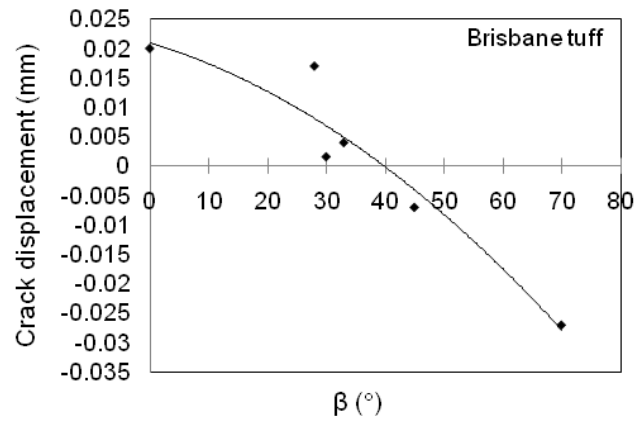


Figure 5.15 Maximum notch crack displacement under mixed-mode I–II loading for Brisbane tuff CCNBD samples

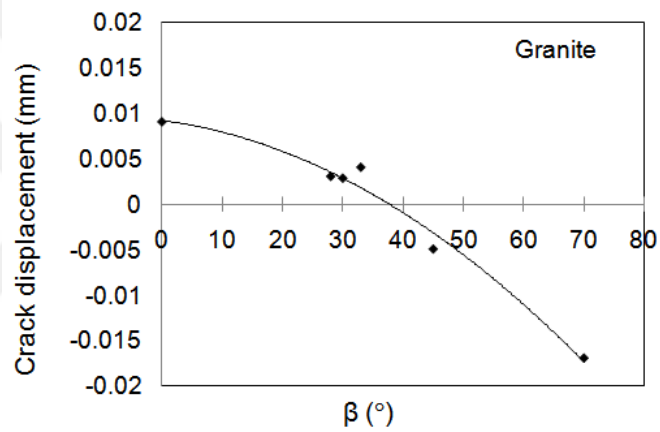


Figure 5.16 Maximum notch crack displacement under mixed-mode I–II loading for granite CCNBD samples

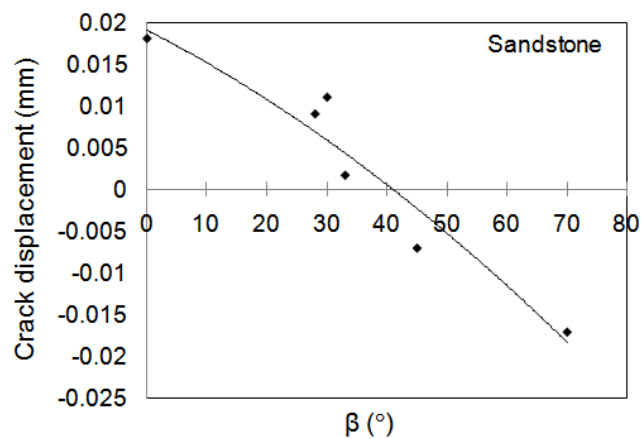


Figure 5.17 Maximum notch crack displacement under mixed-mode I–II loading for sandstone CCNBD samples

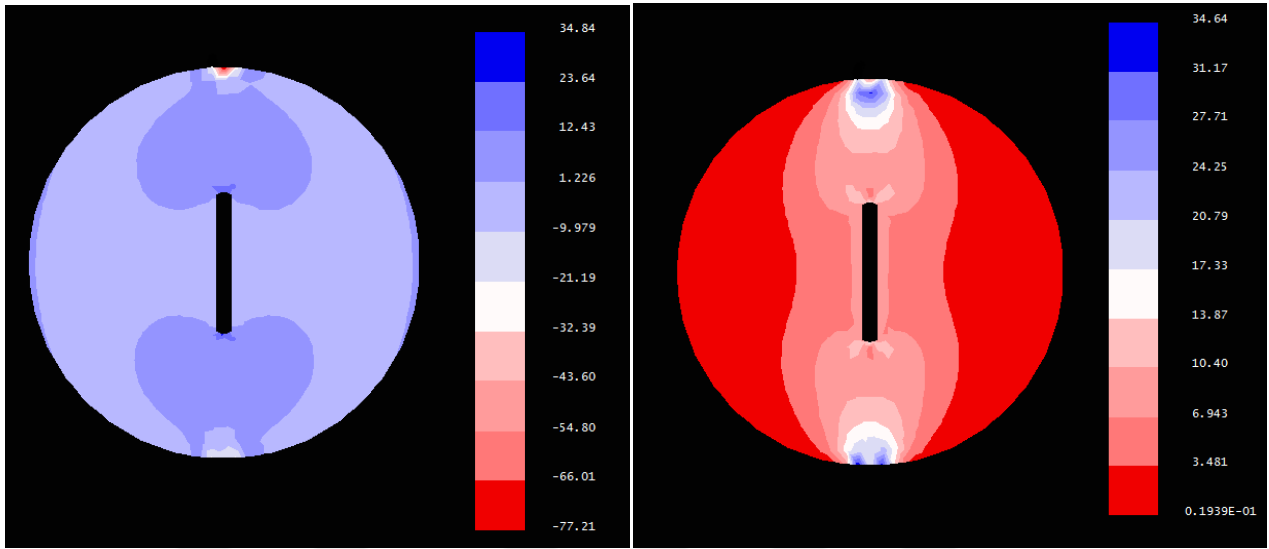
5.2.3 Numerical simulation results

Series of finite element modellings were conducted to model notch crack initiation and propagation under diametral loading. The stress distribution and crack propagation analysis in the specimen were done using the finite element program, FRANC2D (FRacture ANalysis Code). FRANC2D was originally developed by Wawrzynek and Ingraffea (1987) at Cornell University and was accepted as a step in developing discrete fracture analysis programs. The crack propagates in the direction predicted using any of the three propagation theories implemented in FRANC2D. These are the maximum tangential stress theory (Erdogan & Sih, 1963), the maximum energy release rate theory (Hussain & Pu, 1974), and the minimum strain energy density theory (Sih, 1974).

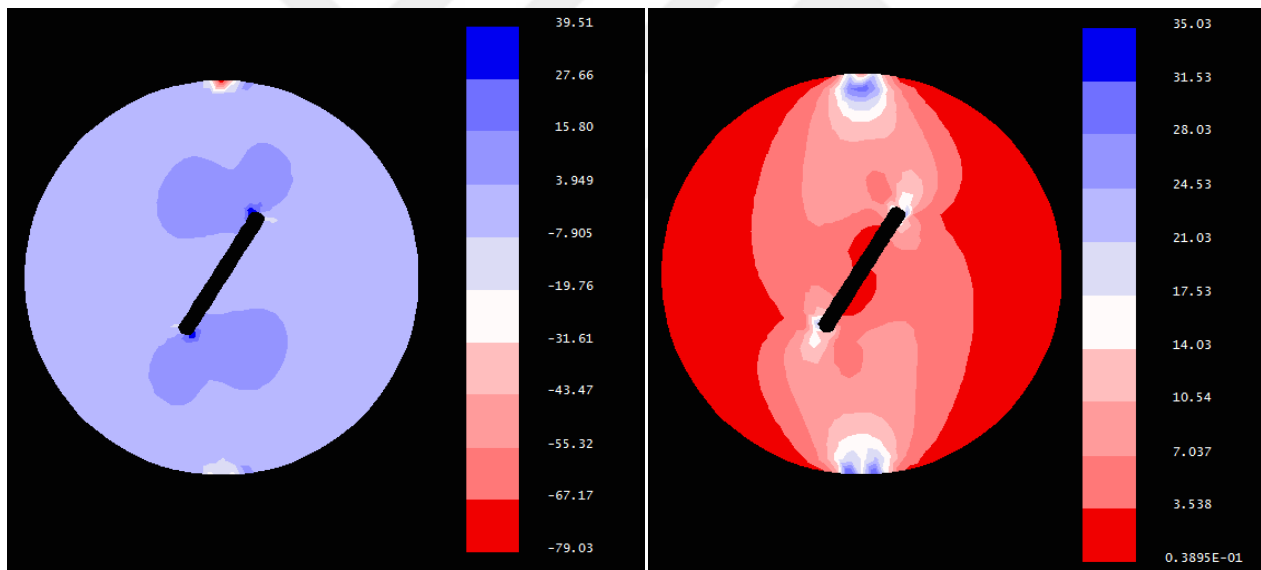
The numerical model was built on the geometry of a CSTBD specimen for which the parameters are diameter, $D = 52$ mm; thickness, $B = 25$ mm and notch crack length, 18 mm. All of the elements used in the finite element models have elastic–isotropic material properties that were defined using the results obtained from the rock characterisation tests. Diametral compressive loads used in the numerical models were based on the ultimate loads obtained experimentally for various crack inclination angles individually, e.g., 4.4 kN for $\beta = 0^\circ$ inclined crack, 5.3 kN for $\beta = 33^\circ$ inclined crack. The bottom of the sample disc was fixed in both x and y directions. The problem type was plain stress and the crack propagation was done using the ‘automatic noncohesive crack propagation’ option of FRANC2D. However, the cohesive crack model implemented in FRANC2D is for mode I fracturing only. Hence, noncohesive crack propagation was used in our models for the sake of accuracy of the inclined crack propagation simulations.

The first numerical modelling series was done for stress distribution analysis around a notch crack in a CCNBD specimen. The loading situations for various inclination angles were simulated properly without any new crack initiation or propagation to eliminate use of the mixed-mode fracture criterion (Figure 5.18). In general, the region of tensile stress distribution extends closer to the centre of the chevron notch crack with increasing crack inclination angles. The maximum tensile stress was obtained along the crack plane at the tip of the $\beta = 0^\circ$ inclined chevron notch. The detailed stress analysis around the face and tip of the notch crack is given in Figure 5.20, Figure 5.21 and Figure 5.22. In contrast, the maximum shear stress was found just under the loading boundary where the high compressive region occurred and the region of shear stress moved from the side of the end of the notch crack to the centre of the end of the notch crack after 30° crack

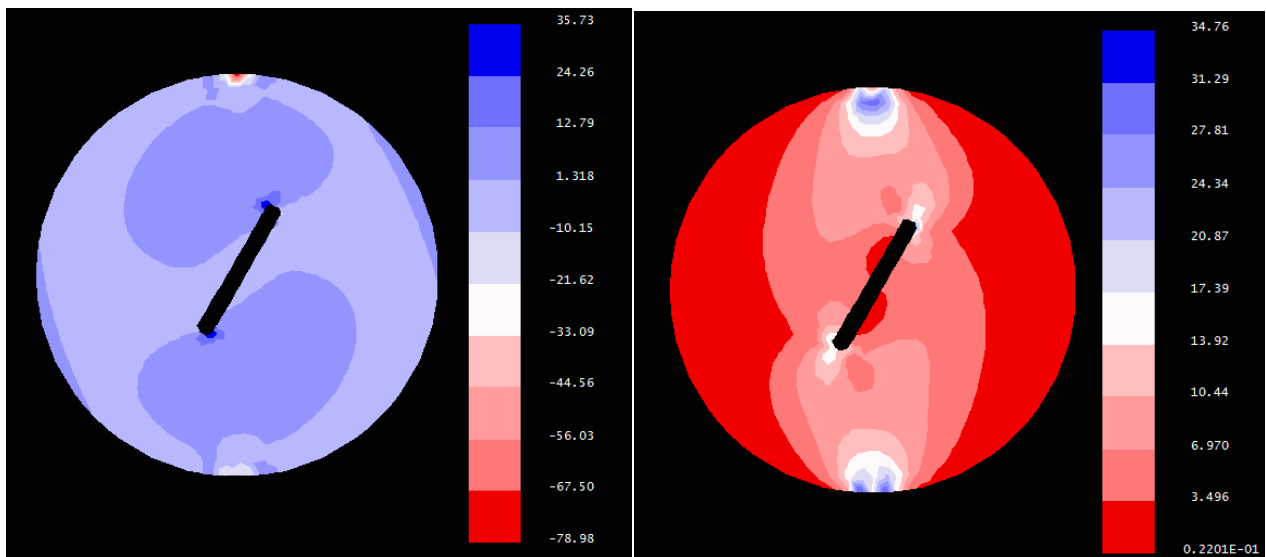
inclination angle. The maximum tensile stresses were obtained at the tip of the notch crack in normal direction to the crack plane with $\beta = 28^\circ$, $\beta = 30^\circ$ and $\beta = 33^\circ$ inclined notch cracks.



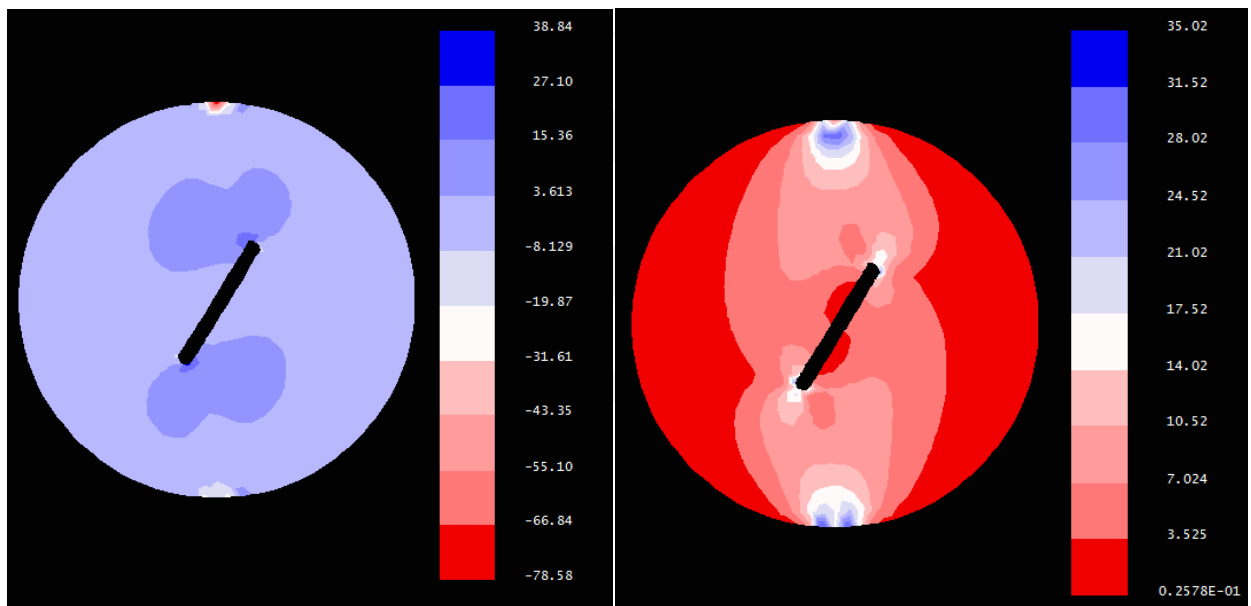
(a)



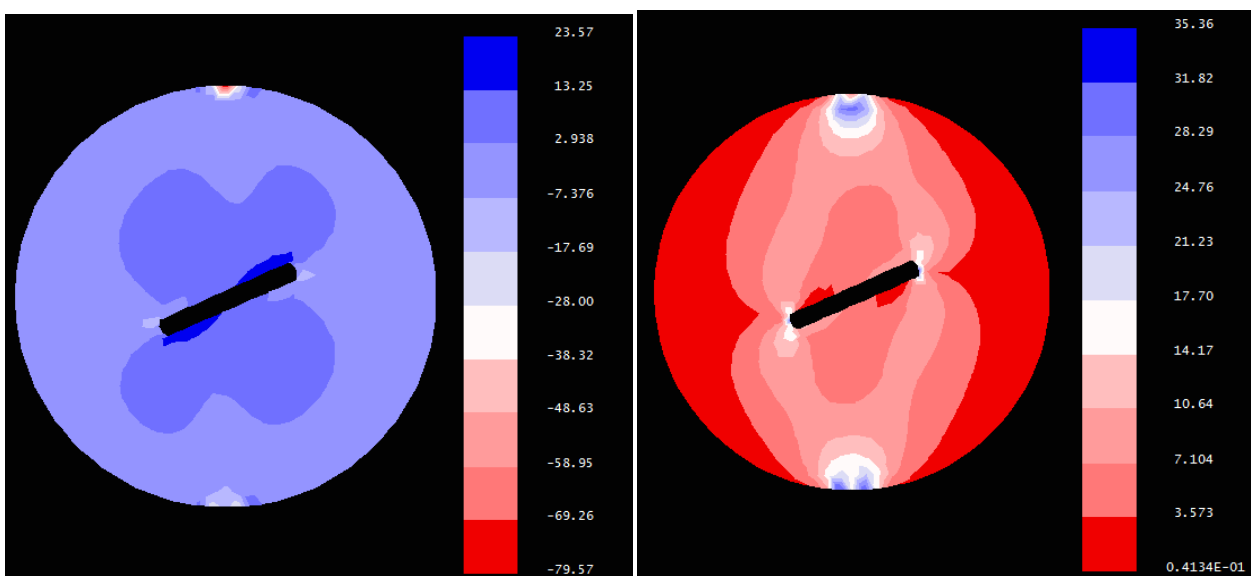
(b)



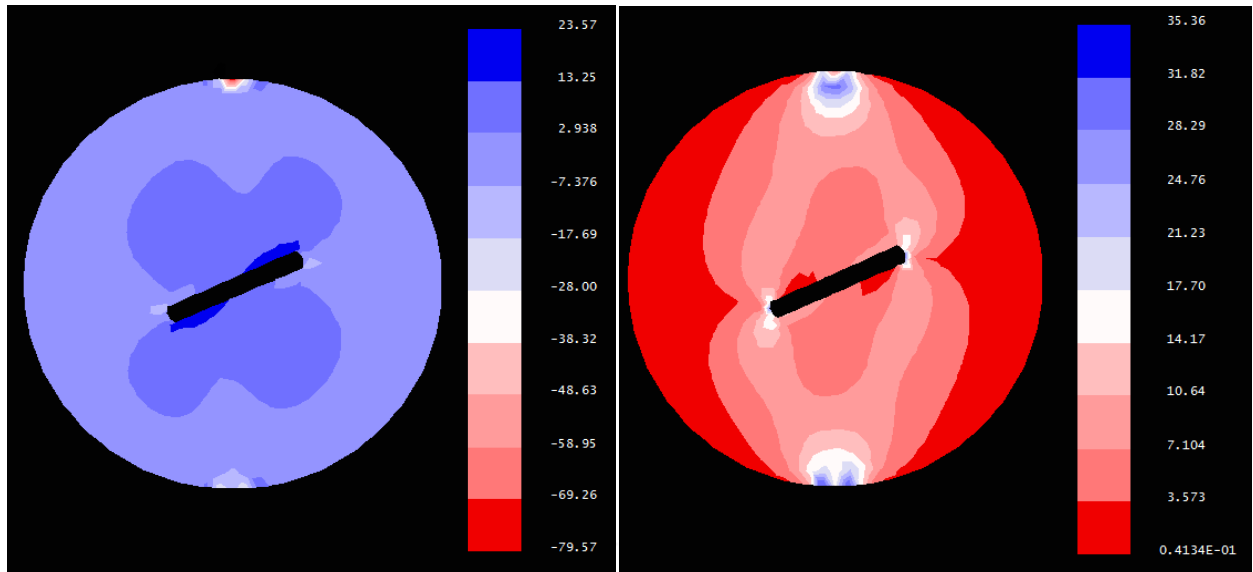
(c)



(d)



(e)



(f)

Figure 5.18 Stress distribution around the notch crack; left-hand side is maximum principal stress (MPa) σ_1 (+: tension; -: compression), right-hand side is shear stress $\tau_{\max}$ (a) $\beta = 0^\circ$ (b) $\beta = 28^\circ$ (c) $\beta = 30^\circ$ (d) $\beta = 33^\circ$ (e) $\beta = 45^\circ$ (f) $\beta = 70^\circ$

In general, the results of the numerical stress distribution analyses show that it is not possible to obtain pure mode I and pure mode II fracturing modes with uniform diametral compressive loading on to the CCNBD geometry. It was found that both normal stress (tension) and shear stress are effective with all inclination angles. Further, normal stress (tension) values are higher than the tensile strength of the material. Thus, it is hard to determine which fracturing mode acts in crack propagation first.

In addition to the contour plots for stress distribution, the ‘line plot’ postprocess option of FRANC2D is used to analyse stress distribution around the notch crack in detail. We named the position at the left-hand side of the notch crack the ‘upper face of crack’ and at the right-hand side the ‘lower face of crack’ according to the diametral compressive loading and inclination of the notch crack (Figure 5.19). The upper end of the line represents the start point of the ‘position on line’ axis in the plots (Figure 5.19).

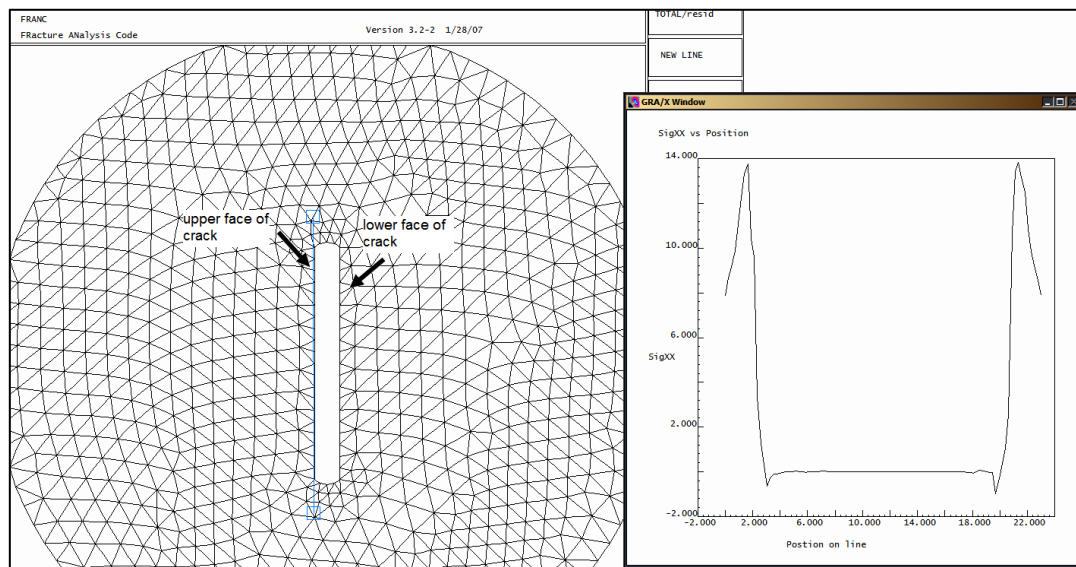


Figure 5.19 Line plot post-process option of FRANC2D for stress distribution analysis around notch crack

Stress distribution around the tip of a notch crack best can be explained by plotting the normal stress, shear stress, and stress in the x-axis separately along the upper face, lower face and at the tip of the notch crack. Figure 5.20 shows the normal stress distribution around the upper face of a notch crack inclined at various inclination angles (β). The normal stress is tensile at the upper face of a notch crack inclined up to $\beta = 33^\circ$. In contrast, normal stress is compressive at the upper face of a notch crack with inclination angles $\beta = 45^\circ$ and $\beta = 70^\circ$ (Figure 5.20). Figure 5.21 shows the normal stress distribution around the lower face of a notch crack with symmetrical results to those of the upper face. These results show points where wing crack initiations take place around the tip of the notch crack. However, normal stress is always compressive around both the lower and upper faces of a notch crack with inclination angles $\beta = 45^\circ$ and $\beta = 70^\circ$ (Figure 5.20 and Figure 5.21). Figure 5.22 shows the normal stress distribution at the tip of a notch crack. The normal stress is tensile for $\beta = 0^\circ$ inclined notch crack, which is a mode I (tensile mode) crack. This result shows that the tensile stress area is moving from the tip of the notch crack through the face of the notch crack.

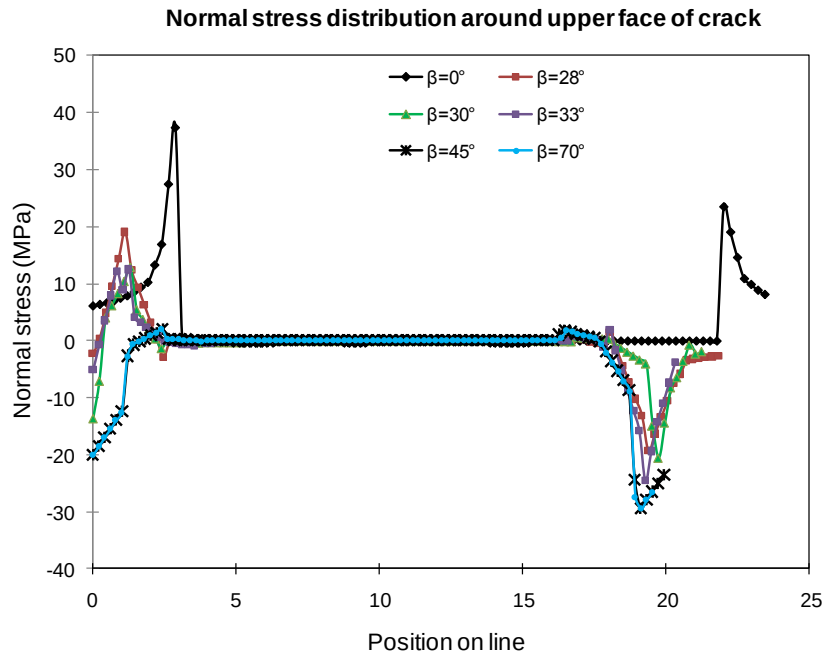


Figure 5.20 Normal stress (σ_n) distribution around the upper face of a notch crack (Tensile: +; Compressive: -)

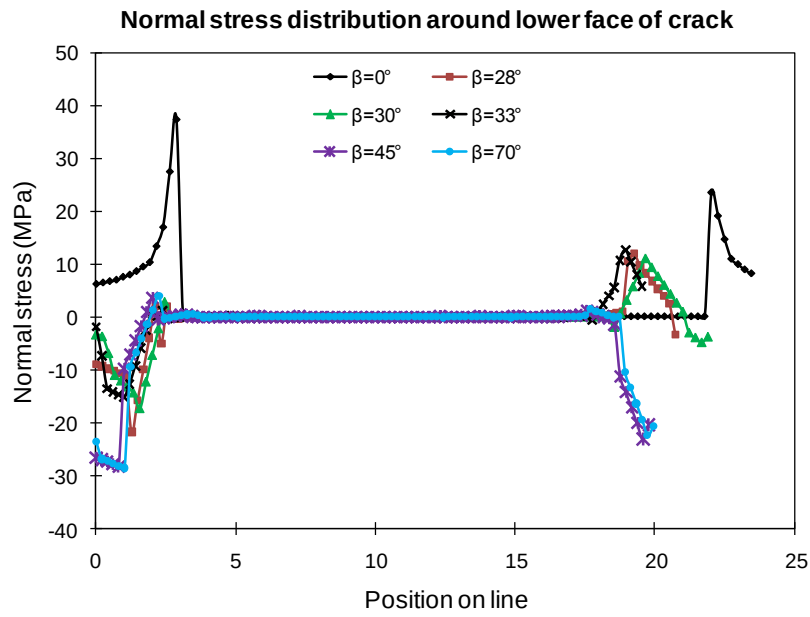


Figure 5.21 Normal stress (σ_n) distribution around the lower face of a notch crack (Tensile: +; Compressive: -)

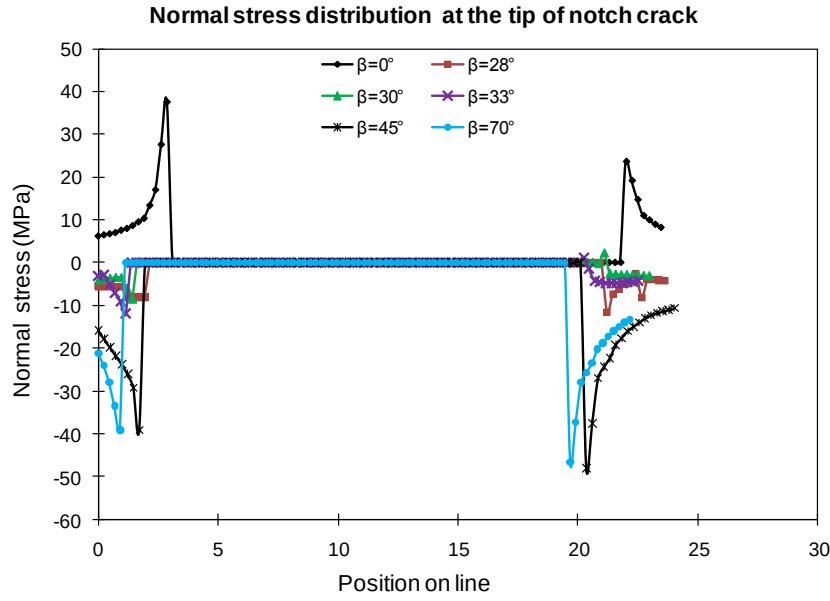


Figure 5.22 Normal stress (σ_n) distribution at the tip of a notch crack (Tensile: +; Compressive: -)

Shear stress distribution around a notch crack is given in Figure 5.23, Figure 5.24 and Figure 5.25. The highest shear stress values are obtained with $\beta = 28^\circ$, $\beta = 30^\circ$ and $\beta = 33^\circ$ inclined notch cracks whereas the lowest shear stress is obtained with $\beta = 0^\circ$ inclined notch crack, as is expected. It is interesting that the value of shear stress, ~ 15 MPa around the upper and lower faces of a crack, is almost equal to the relevant tensile stress for $\beta = 28^\circ$, $\beta = 30^\circ$ and $\beta = 33^\circ$ inclined notch cracks. The shear stress distribution at the tip of a notch crack is shown in Figure 5.25. In our sign convention, tension is positive and compression is negative; hence, negative shear is possible around the upper and lower faces of notch cracks but not at the tip of a notch crack.

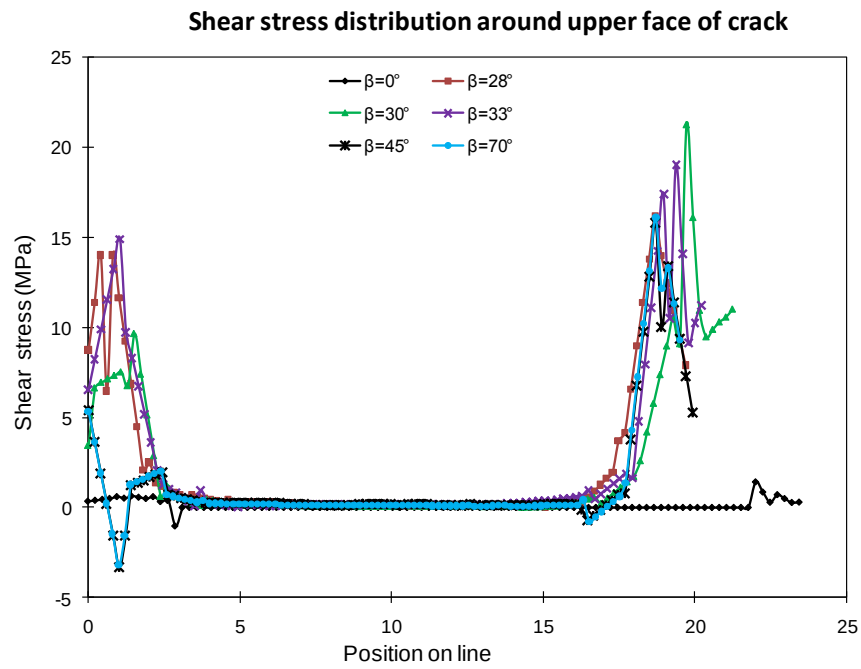


Figure 5.23 Shear stress (τ) distribution around the upper face of a notch crack

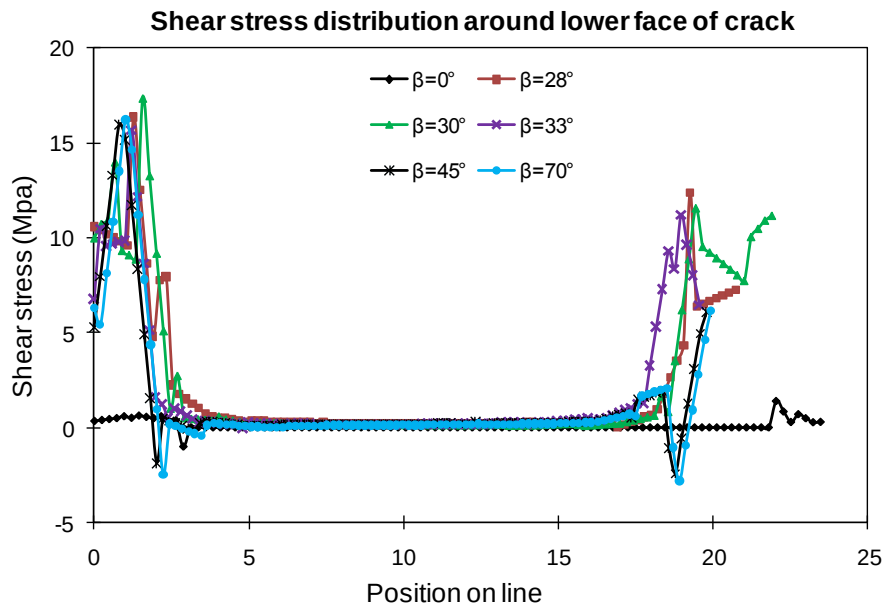


Figure 5.24 Shear stress (τ) distribution around the lower face of a notch crack

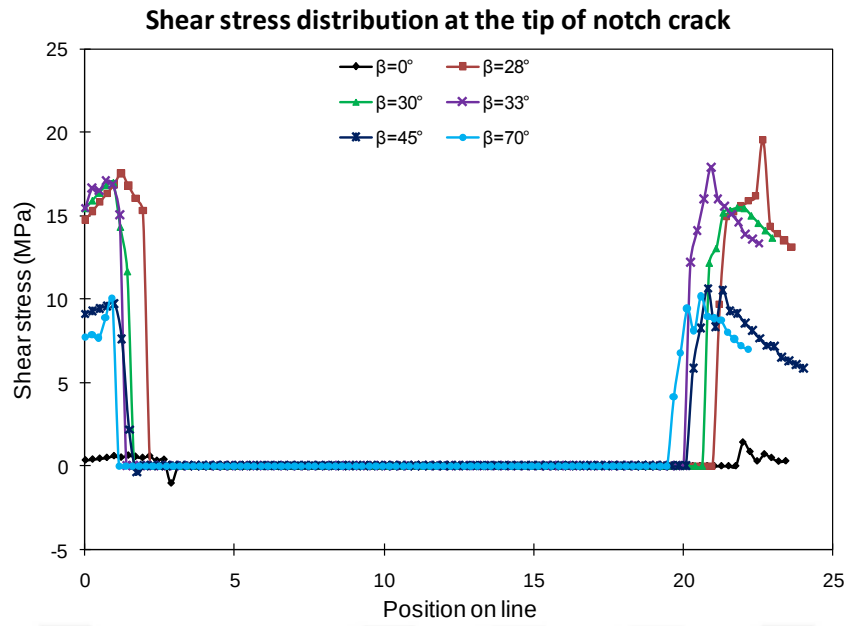


Figure 5.25 Shear stress (τ) distribution at the tip of a notch crack

Stress distribution in x direction (σ_{xx}) around the upper and lower faces of a notch crack is given in Figure 5.26. The extended tensile stress distribution through to the centre of the notch crack is obvious for all inclined cracks with an angle except $\beta = 0^\circ$ inclined notch crack. The plot, shown in Figure 5.26, shows that the extension of the tensile stress region through to the centre of notch crack gets larger with increasing crack inclination angles.

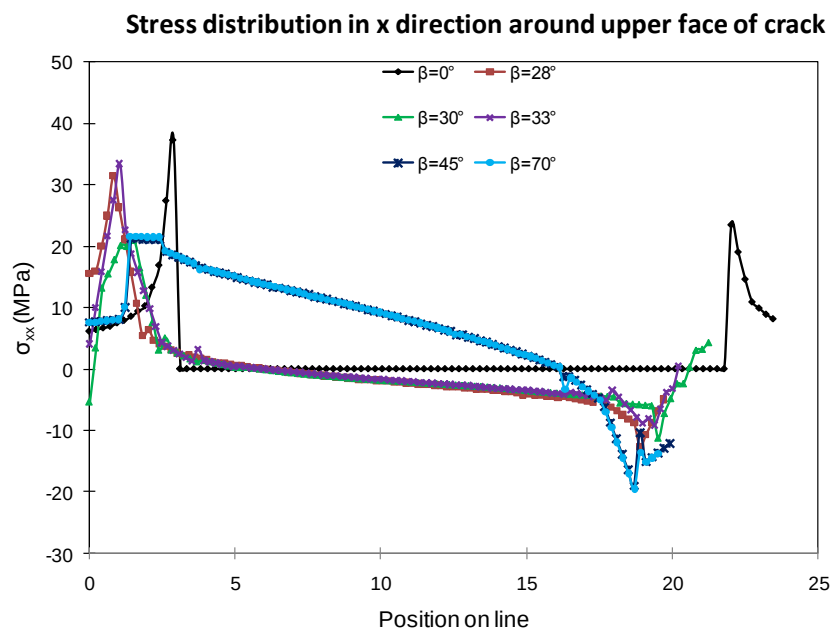


Figure 5.26 σ_{xx} distribution around the upper face of a notch crack (Tensile:+; Compressive: -)

—)

After the stress distribution analysis, crack propagation analyses were done to investigate the crack displacements depending on the crack inclination angles. FRANC2D was accepted as a step in developing the discrete fracture analysis programs. During performance of a discrete crack analysis for crack growth, not only the geometry of the crack is represented explicitly at each step, but also the mesh must be modified at each step to reflect the current crack configuration. The automatic re-meshing strategy adopted in FRANC2D is to delete the elements in the vicinity of the crack tip, move the crack tip, and then insert a trial mesh to connect the new crack to the existing mesh.

The crack propagation simulation results are shown in Figure 5.27. It was found that crack initiation angle was strongly dependent on the notch crack inclination angle. Moreover, the direction of crack propagation was found to be parallel to the loading direction and is in good agreement with experimental findings. Because the high tensile stress zone was observed to extend along the centre of the notch crack instead of the tip of the notch crack with increasing inclination angle in the stress distribution analysis in the previous section, crack initiation in crack simulations starts from a region closer to the centre of the notch crack when the β increases.

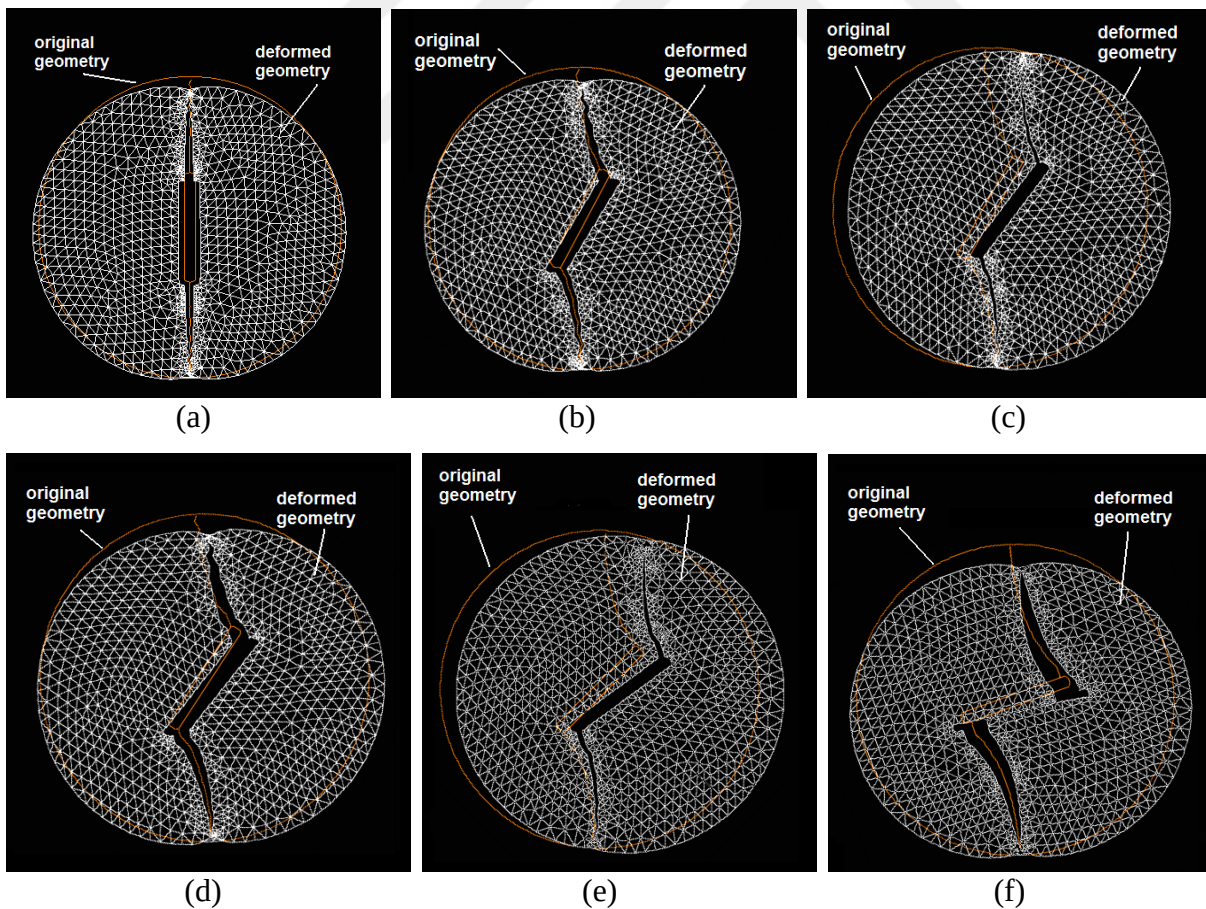


Figure 5.27 Simulated original and deformed notch cracks with various inclination angles (a)

$\beta = 0^\circ$ (b) $\beta = 28^\circ$ (c) $\beta = 30^\circ$ (d) $\beta = 33^\circ$ (e) $\beta = 45^\circ$ (f) $\beta = 70^\circ$

A comparison between mode I stress intensity factor and mode II stress intensity factor calculated by FRANC2D is given in Figure 5.28. Mode I stress intensity factor was found to be always higher than mode II stress intensity factor at the tip of the cracks inclined at various angles. The highest mode I and mode II stress intensity factor values were obtained with 30° and 33° inclined crack simulations, as in the experimental results. Because classical fracture criteria are implemented in FRANC2D, the calculated mode II stress intensity factor values are expected to be smaller than mode I stress intensity factor values. According to the maximum stress criterion (Erdogan & Sih, 1963), the mode II maximum stress intensity factor reaches its critical value (K_{IIc}) when the mode I stress intensity factor equals zero ($K_{II\max} = K_{IIc}$, $K_I = 0$) for pure shear loading. However, one of the most important mixed-mode criteria, maximum tangential stress, was derived for a crack under monotonic tension. Hence, the predicted K_{IIc} values by the maximum tangential stress criterion are smaller than those of K_{Ic} . However, for brittle rocks, shear resistance has been accepted as greater than the tensile strength, thus, K_{IIc} should be larger than K_{Ic} of rocks. In some research, it has been found that mixed-mode fracture criteria may not be capable of predicting mode II fractures or for measuring mode II fracture toughness (Backer, 2004; Rao *et al.*, 2003; Whittaker *et al.*, 1992).

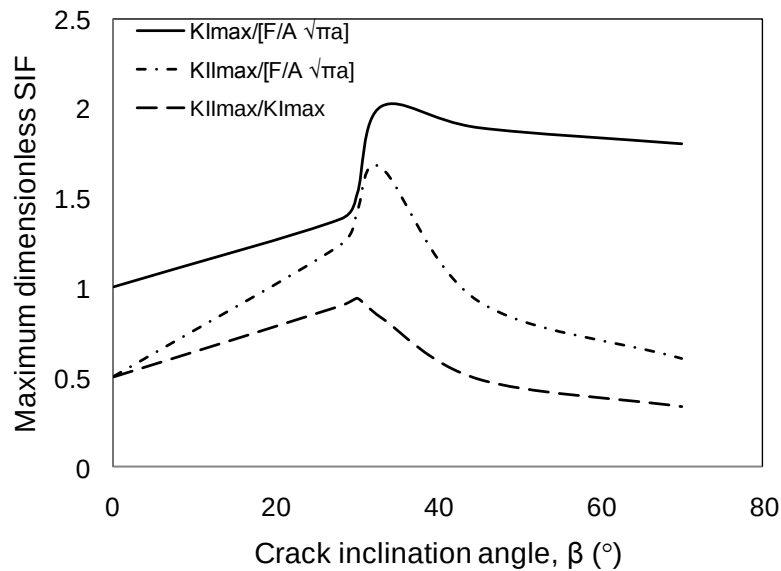


Figure 5.28 Mode I and mode II stress intensity factor values for a crack tip inclined at various angles

5.3 Discussion of Fracture Toughness Results

A wide variety of specimen types and methods are employed in fracture toughness calculations of rocks, which result in scattered values for the same rock type. In general, for the same rock, higher fracture toughness values are determined with methods having chevron notches and lower fracture toughness values are determined with bending methods (Chang *et al.*, 2000). Two testing methods, SR and CCNBD, suggested by the ISRM (1995) were used to calculate mode I fracture toughness of the Brisbane tuff. It was found that mode I fracture toughness determined by CCNBD is smaller than that determined by the SR method.

5.3.1 Comparison of CCNBD and SR methods

The fracture toughness values obtained by the SR method are higher compared with the fracture values obtained by the CCNBD method. This research determined the critical stress intensity factor values ($Y_{\min}^*$) of CCNBD specimens, the value of which is a crucial factor in the suggested method proposed by ISRM (1995) for mode I fracture toughness determination. Wang *et al.* (2003) questioned the value of that key factor, stress intensity factor, for CCNBD specimens. They tried to calibrate the minimum stress intensity factor of CCNBD specimens. They concluded that the $Y_{\min}^*$ value calculated using the formula given by ISRM (1995) is too small compared with their calibrated value (Wang *et al.*, 2003). Our results in this research confirm Wang *et al.*'s (2003) findings. We found the $Y_{\min}^*$ value of 0.98 using Eq. (5.2) given by ISRM (1995) and the calculated fracture toughness of Brisbane tuff to be $1.48 \text{ MPa}\sqrt{\text{m}}$. In contrast, we found the $Y_{\min}^*$ value to be 1.21 using the STCA method (Shetty *et al.*, 1985) and the relevant fracture toughness value of the Brisbane tuff to be $1.74 \text{ MPa}\sqrt{\text{m}}$. It is concluded that further work on a stress intensity factor analysis of CCNBD specimens is definitely required. The accuracy of the method can be further improved if accurate and compliant results from numerical and analytical methods are available.

In contrast, the fracture toughness values obtained by the SR method have been found to be overestimated when compared with the fracture toughness of rocks found with CCNBD within the same ranges of uniaxial compressive strength (120–140 MPa) and uniaxial tensile strength (12–14 MPa) (Guo *et al.*, 1993; Whittaker *et al.*, 1992; Zang, 2002). In order to check the validity of fracture toughness values obtained by CCNBD, the relationship between fracture toughness and the physico-mechanical properties of rocks has been considered. One might expect that the mode I

fracture toughness, in which the rock is separated by a tensile stress, would be related to the tensile strength of rock. According to Whittaker *et al.* (1992) and Zhang (2002), the mode I fracture toughness of rock is directly proportional to its tensile strength. As given by Zhang's (2002) equation for the direct relation between uniaxial tensile strength and fracture toughness of rocks, the expected fracture toughness of Brisbane tuff should be between $1.12\text{--}2 \text{ MPa}\sqrt{\text{m}}$. The fracture toughness from the CCNBD specimens is within this range; in contrast, the fracture toughness values obtained with the SR specimens are out of this range. Similar findings were made by Whittaker *et al.* (1992) for the relationship between fracture toughness and tensile strength of rocks. According to the equation given by Whittaker *et al.* (1992), the expected fracture toughness of Brisbane tuff should be between $1.5\text{--}1.9 \text{ MPa}\sqrt{\text{m}}$. Further, there is a close correlation of mode I fracture toughness with uniaxial compressive strength (Whittaker *et al.*, 1992). According to the derived equation, the expected fracture toughness should be $1.58 \text{ MPa}\sqrt{\text{m}}$ for Brisbane tuff-1 and $1.78 \text{ MPa}\sqrt{\text{m}}$ for Brisbane tuff-2. It should be borne in mind; however, that those kinds of relationships between fracture toughness and physico-mechanical properties were obtained from various specimen configurations and testing techniques. However, fracture toughness results have been shown to be comparable.

As far as a general comparison between the two methods is concerned, the CCNBD method definitely had advantages in terms of simplicity of sample preparation and less material required for testing. It is also unnecessary to perform pre-cracking for a CCNBD specimen because it uses a chevron notch that self-precracks during testing and leads to stable crack propagation. Another remarkable advantage of the CCNBD method over the SR method in terms of precision is higher load capacity and consistent results for each test. In our research, all the CCNBD samples show high precision in maximum load measurements. In addition, it is possible to measure mode I, mode II and mixed-mode I–II fracture toughness by inclining the notch at different angles with respect to the axis of diametral load.

5.3.2 Comparison between experimental and numerical results of mixed-mode fracturing

It was found experimentally that plastic deformation always took place before rupture in all three rock types, during opening of chevron notch cracks up to 33° crack inclination angle. However, there was no plastic deformation observed before rupture with closing of the notch cracks after 33° crack inclination angle. This output may be explained by nonlinear fracture mechanics. According to nonlinear fracture mechanics, a plastic deformation region before failure means that there is a

FPZ in front of the crack tip under tensile stress (mode I loading) and its size depends on the loading condition, e.g., cyclic loading, dynamic impact loading, etc. This concept is consistent with our CMOD observations that there was a closing stress effective on the crack plane after 33°, hence, no further opening stresses to cause the FPZ occur in front of the crack tip.

A diametral compressive load is a kind of compressive-shear loading across a crack plane. However, any shear crack parallel to the crack plane was not observed experimentally with all three rock types up to 70° crack inclination angle, except in the 70° inclined sandstone and granite samples. However, pure shear loading was not found to result in a true mode II fracture in some studies (Huang & Wang, 1985; Swartz & Taha, 1990). For example, Swartz and Taha (1990) and Huang and Wang (1985) found that the central crack that initiated at the notch tip propagates by deviating from the original notch plane at an angle of 50°–70° and the fracture was of mode I in their common K_{IIC} tests.

In order to check the validity of our numerical modelling, a comparison of numerical results and experimental findings about crack extension angles (θ) and crack inclination angles (β) was made (Figure 5.29). Additional crack inclination angles of 20° and 60° were included in the comparison of experimental and numerical outputs by using experimental results from other Brisbane tuff samples beside Brisbane tuff-2 specimens. Good agreement was found between numerical and experimental findings about crack extension angle (θ) and crack inclination angle (β). The crack initiation angle was found to increase with increasing crack inclination angle. In literature, the counter clockwise direction for crack initiation angles is accepted as positive under compression-shear loading. Likewise, the clockwise direction for crack initiation angles is accepted as negative under tension-shear loading. Mostly, crack initiation angles decrease with increasing crack inclination angles under tension-shear loading. In contrast, crack initiation angles increase with increasing crack inclination angles under compressive-shear loading conditions. Both our experimental and numerical results are in good agreement with findings in literature. Once this agreement was obtained with our numerical modelling, it can be decided that FRANC2D is a good numerical tool for mixed-mode crack propagation modelling under compression-shear loading, although we obtained smaller mode I stress intensity factor values than mode II stress intensity factor values.

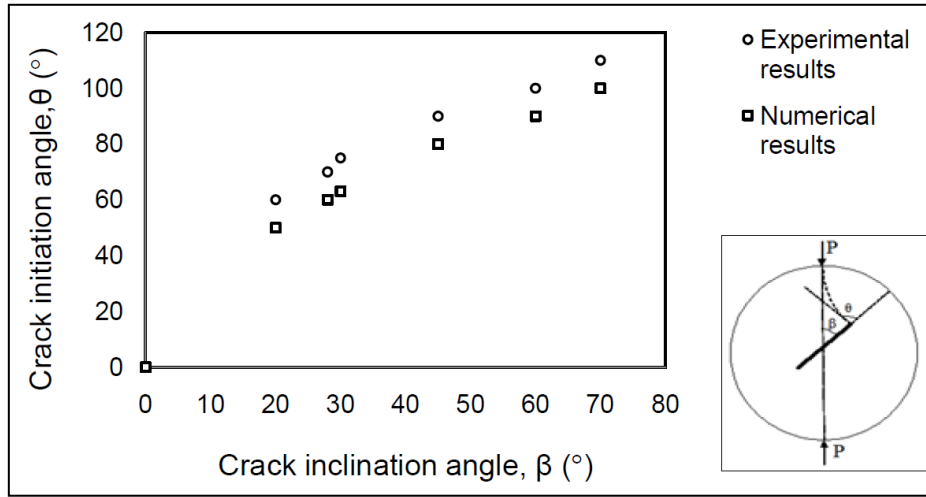


Figure 5.29 Variation of crack initiation angle (θ) with crack inclination angle (β)

Numerical modelling results show that both tensile and shear stresses are acting around the face and at the tip of inclined chevron cracks with all various crack inclination angles. The maximum tensile stress was obtained at the tip of the $\beta = 0^\circ$ inclined chevron notch crack, at 38 MPa. In contrast, the highest shear stress was obtained around the faces of the notch crack with $\beta = 30^\circ$ and $\beta = 33^\circ$ inclined cracks, at 17 MPa.

The state of stress around a pre-existing Griffith crack received attention after Inglis's (1913) study. Ode (1956) made an impressive study on the state of stresses around a crack oriented at an angle to one of the principal stresses. He assumed a plate containing at its centre one elliptical crack loaded along its edge by constant compressive and confining stresses. He found that the normal stress component tangential to the crack closes the crack faces with a 45° inclination angle. His solutions are valid for an elliptical crack under confining stress conditions. However, our results are in agreement with Ode's solutions for a crack having $\geq 45^\circ$ inclination angle under diametral compressive load without confining stress conditions by using normal stress distribution analysis of the upper face of a crack.

The simplest model for such a crack was proposed by Fairhurst and Cook (1966). Fairhurst and Cook (1966) used the inclined crack of a wing crack model as a mechanism to generate local tensile forces (F) normal to the direction of compressive loading. These forces originate from sliding of the opposite sides of the contact area. It is supposed that in brittle rocks a crack grows as a result of concentration of tensile stresses at its tips, i.e., as a mode I crack. Hence, only horizontal projections F of those forces tending to open the crack are taken into account. It should be noted that it is the contact area that produces these wedging forces, F . Without the contact, force F would be equal to

zero. However, an initial crack with contacting faces is only an extreme case of a defect capable of transferring the applied compression into local tension (Germanovich *et al.*, 1994).

5.4 Chapter Summary

In order to determine the mode I fracture toughness of Brisbane tuff, two ISRM suggested methods (ISRM, 1981; 1995), SR and CCNBD, were used. It was found that CCNBD has more advantages over the SR method in terms of sample preparation and failure load consistency. The fracture toughness values obtained by the SR method were higher compared with the fracture values obtained by the CCNBD method. To confirm the validity of those methods, the results were compared with some relationships between fracture toughness and rock physical properties. Also, analytical analysis was done to find the most accurate dimensionless stress intensity factor to calculate fracture toughness of Brisbane tuff.

Disc specimens of three rock types, Brisbane tuff, granite and sandstone, were diametrically loaded with various angles (β) inclined relative to the diametral compressive loading direction to investigate mixed-mode rock fracturing. The highest failure load was obtained with a 33° inclined notch crack in all three rock types. Also, typical wing cracks were seen at the tip of all the notch cracks inclined at various angles from $\beta = 28^\circ$ to $\beta = 70^\circ$ in all three kinds of CCNBD rock specimens. However, no typical shear crack was found parallel to the notch crack plane. It was concluded that it may not be possible to get pure shear fracturing with the CCNBD specimen geometry under diametral compressive loading with any notch crack inclination angle.

Experiments performed on diametrically loaded CCNBD specimens have showed that cracks initiated at the tip of the notch for all three kinds of rock specimens with values of β up to 30° only. However, when the inclination angle is bigger than 33° , the crack initiation point moved from the tip to the centre of the notch crack as the inclination angles increased.

Numerical modelling studies showed that stress intensities at the tip of the notch crack were strongly dependent on the notch crack inclination angle. The high tensile stress concentration region tends to extend from the tip to the centre of the notch crack when the inclination angle increases. This result is consistent with the experimental findings about crack initiation points along the inclined notch cracks. Moreover, numerical modelling results show that both tensile and shear stresses are effective at the face and tips of the inclined chevron cracks for all various crack

inclination angles. The maximum tensile stress was obtained at the tip of the $\beta = 0^\circ$ inclined chevron notch crack. The obtained maximum tensile stress is 38 MPa, higher than the indirect tensile strength of the rock. The highest shear stress was obtained with $\beta = 30^\circ$ and $\beta = 33^\circ$ inclined cracks, at 17 MPa. In contrast, the smallest shear stress was obtained with $\beta = 0^\circ$ inclined crack.

After crack propagation modelling, it can be concluded that the crack initiation angle (θ) is a function of the crack inclination angle (β). Good agreement was found between numerical and experimental findings for both crack initiation angle (θ) and crack inclination angle (β). It was found that crack initiation angle increases with increasing crack inclination angle under compression-shear loading configuration.



CHAPTER 6

DEGRADATION OF INDIRECT TENSILE STRENGTH OF ROCKS THROUGH CYCLIC LOADING

This chapter presents the results of laboratory experiments during the investigation of stress–strain characteristics of Brisbane tuff Brazilian disc specimens under diametral compressive cyclic loading. Two different cyclic loading methods were used, namely sinusoidal cyclic loading and cyclic loading with increasing mean level; the first method obtained the S–N curve approach to the indirect tensile strength of rocks for the first time in literature and the second method investigated the effect of increasing cyclic loading on the indirect tensile strength of rocks.

Brisbane tuff Brazilian disc specimens were used to simulate indirect tensile loading, as is usually done in rock mechanics to indirectly measure tensile strength of rocks (the so-called Brazilian indirect tensile test).

6.1 Sinusoidal Cyclic Tests

Sinusoidal cyclic loading test series were carried out to obtain S–N curve illustrative of the continuous weakening of rock with the increase in number of cycles (N) required for failing a specimen loaded to a certain upper peak stress (S). This research reports for the first time in literature the S–N curve for indirect tensile strength degradation due to fatigue under cyclic loads. Both the amplitude and the mean value of the loading cycles are constant in sinusoidal cyclic loading (Figure 6.1).

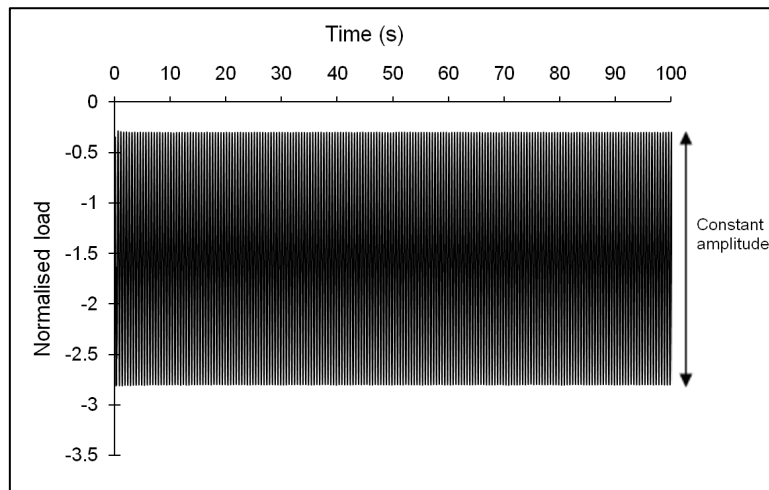


Figure 6.1 Sinusoidal cyclic loading

During sinusoidal cyclic tests, the machine is kept running in a particular stress range, alternating the stress until the specimen fails. The number of cycles of stressing up to failure is recorded. The test is repeated for different load ranges. Such tests enable S–N graphs to be plotted. The vertical axis is the stress amplitudes, and the horizontal axis is the number of cycles N to failure. The number of cycles to stress reversal that a specimen can sustain before failure occurs depends on the stress amplitude; the bigger the stress amplitude the smaller the number of cycles that can be sustained. The fatigue limit is the stress amplitude at a particular number of cycles that will not result in failure, in other words the material will endure an infinite number of cycles with smaller stress amplitudes.

6.1.1 Determination of S–N curve for the fatigue limit of indirect Brazilian tensile strength of Brisbane Tuff

All the diametral compressive cyclic tests were performed with Instron testing machines with a 100 kN load cell. During the monotonic test series on Brazilian disc specimens, it was found that the standard Brazilian indirect tensile test caused catastrophic crushing failure of the disc specimens, rather than the expected tensile splitting failure initiated by a central crack. This finding led to an investigation of the effect of loading conditions on the failure of Brazilian disc specimens by using three steel loading arcs of different angles applied to three different rock types. However, because there is no international standard and formulation for determining the indirect tensile strength of a rock specimen under diametral loads, the standard Brazilian jaws proposed by ISRM (1978) were used in cyclic loading tests.

The tests were carried out on Brisbane tuff, which is the main host rock of Brisbane's first motorway tunnel, the CLEM7, from which core samples were obtained. Test specimens were prepared from standard Brazilian discs with a diameter of 52 mm and thickness of 26 mm. To perform cyclic tests for obtaining S–N curve, the maximum applied stress level was reduced from 100% at 5% intervals until the specimen did not fail.

The first important conclusion drawn from the sinusoidal cyclic tests is that the failure load used in calculations of indirect tensile strength is definitely weakened 30% by repetitive loading. Results are given in Table 6.1.

Table 6.1 Results of sinusoidal cyclic loading

$S_{\max}$ (as % of static failure load)	$F_{\max}$ (failure load, kN)	Number of cycles to failure (N)	Percentage decrease in failure load
100	23	1	0
95	21.8	2	5
90	20.7	98	10
90	20.7	268	10
90	20.7	104	10
90	20.7	190	10
80	18.4	3456	20
80	18.4	775	20
80	18.4	18723	20
80	18.4	121000	20
80	16.1	12047	30
70	16.1	74515	30
70	16.1	500000	30
70	16.1	560000	30
70	16.1	800000*	30
68	16.0	800000*	30

*No failure

Figure 7.2 shows the S–N curve for indirect tensile strength of Brisbane tuff. The indirect tensile strength of Brisbane tuff was determined as 10.8 MPa under monotonic loading. After the sinusoidal cyclic tests, it was observed that the indirect tensile strength of Brisbane tuff was reduced from 10.8 MPa to 7.5 MPa as a result of the cyclic loading. The percentage at which the specimen did not fail even with 800000 cycles was considered to be the fatigue stress level of indirect tensile strength.

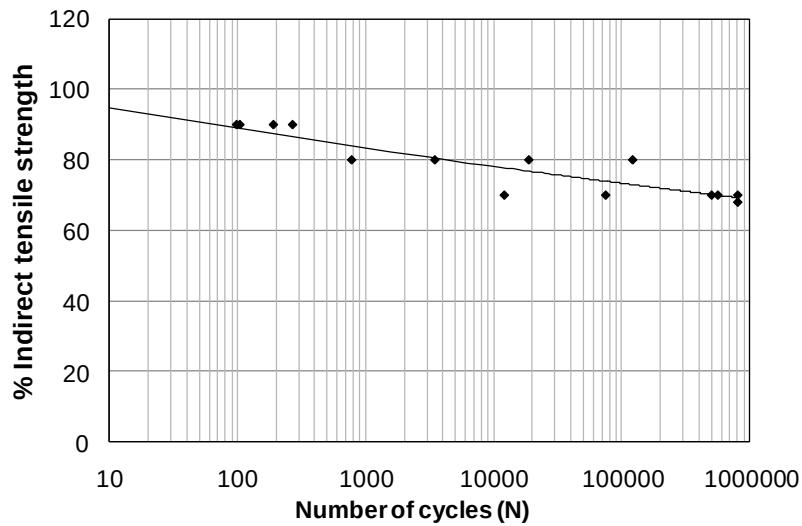


Figure 6.2 S—N curve for indirect tensile strength of Brisbane tuff

6.2 Effect of Increasing Cyclic Loading on Indirect Tensile Strength of Brisbane Tuff

The cyclic loading used in the first of two different series of tests was ramp waveform cyclic compressive loading with increasing mean level, termed increasing cyclic loading. The loading frequency was 1 Hz for all tests. The amplitude is expressed as absolute ($\pm$ value) equal to the total range. An illustration of dynamic cyclic loading is given in Figure 6.3. Four different amplitudes were chosen to investigate the effect of fatigue on the indirect tensile strength of Brisbane tuff; 2.3 kN, at 10% of the Static Ultimate Load (SUL); 4.6 kN (20% SUL); 6.9 kN (30% SUL) and 9.2 kN (40% SUL).

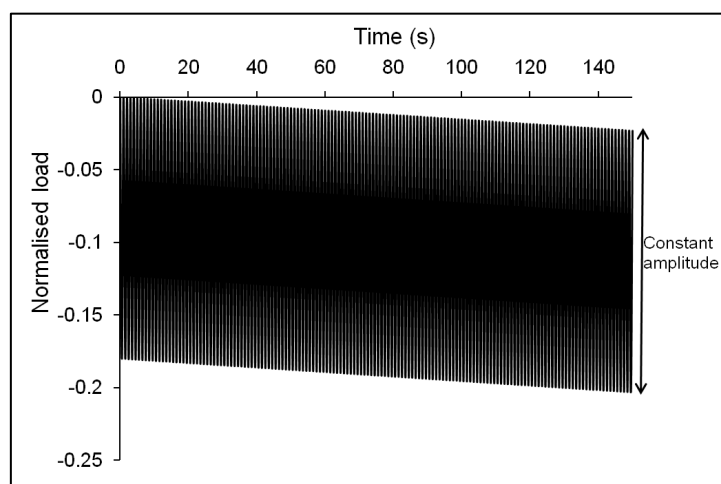
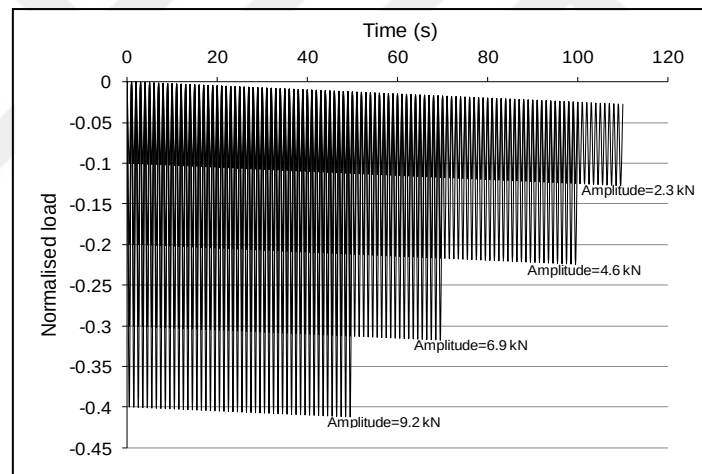


Figure 6.3 Load–number of cycles graph illustrating increasing cyclic loading

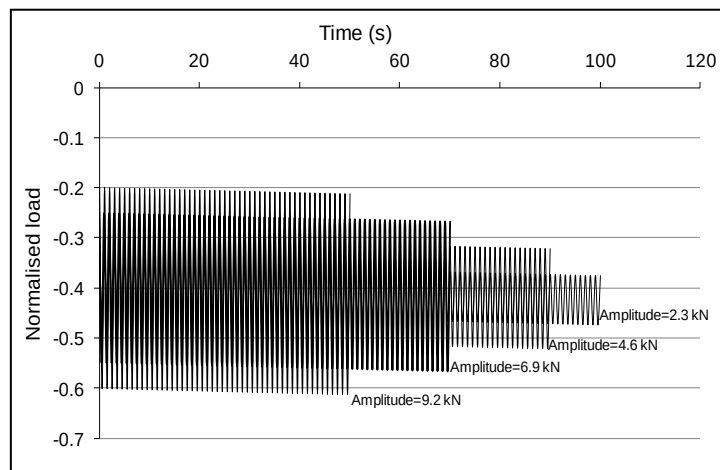
Because the extensometer installed in the Instron is not very sensitive for efficient dynamic strain reading, special DAQ software called EDAP was designed for cyclic tests on Brazilian disc

samples. The program acquires four channels of data: load, displacement and axial/lateral strain readings. The Brazilian disc sample has two strain gauges attached to it on faces perpendicular to each other, lateral and axial to the force applied to the sample.

Two types of increasing cyclic loading test series were performed on Brisbane tuff specimens: (1) load cycles with the same increasing valley values (termed Type I; Figure 6.4a) and (2) load cycles with the same increasing mean values (termed Type II; Figure 6.4b). With Type I cyclic loading, loading cycles have increasing mean stress values with the same valley values for all four different amplitudes. In contrast, loading cycles have increasing valley value sets at each different amplitude value with the same mean stress with Type II cyclic loading. The main purpose of performing two types of increasing cyclic loading is to investigate if there is a significant difference in fatigue damage mechanism caused by those two types of cyclic loading depending on the valley value at which the unloading of each cycle stops. It was believed that the fracturing energy balance in specimen depends on the stress level where the unloading stops.



(a)



(b)

Figure 6.4 Increasing cyclic loading with (a) same valley and (b) same mean values

The series of Type I diametral compressive dynamic cyclic loading tests was performed on 12 CCNBD Brisbane tuff samples. The analyses of these tests focused on the reduction of Indirect Tensile Strength (ITS) under dynamic cyclic loading depending on different amplitudes. Indirect tensile strength of Brisbane tuff under static loading was calculated as 10.8 MPa using ISRM (1978) suggested methods. The indirect tensile strength of Brisbane tuff was reduced from 10.8 MPa to 7 MPa as a result of the cyclic loading. The reduction of ITS because of cyclic loading is shown in Table 6.2 by comparing with static ITS values.



Table 6.2 Results of Type I cyclic loading tests performed on Brazilian disc specimens

Sample	Amplitude % SUL	Ultimate load (kN)	Number cycles up to failure	Indirect tensile strength (ITS)	Reduction of ITS (%)
NST207- Rp1	10	17.4	2583	8.2	24.1
NST207- Rp2	10	15.6	2384	7.3	32.5
NST207- Rp3	10	18.1	2723	8.5	21.3
Average		17.03	2563	8.0	25.9
NST207- Rp1	20	17.5	2195	8.2	24.1
NST207- Rp2	20	15.4	1887	7.2	33.4
NST207- Rp3	20	14.6	1679	6.8	37.1
Average		15.9	1920	7.4	31.5
NST207- Rp1	30	17.3	1906	8.1	25.0
NST207- Rp2	30	15.2	1334	7.1	34.3
NST207- Rp3	30	16.1	1556	7.5	31.0
Average		16.2	1598	7.6	30.1
NST207- Rp1	40	14.0	738	6.5	39.9
NST207- Rp2	40	14.4	992	6.7	38.0
NST207- Rp3	40	16.8	1227	7.9	27.0
Average		15.0	985	7.0	35.0

To date, various experimental observations of cyclic loading tests have indicated that dynamic cyclic loading seems more effective for ITS degradation of rocks than sinusoidal compressive cyclic loading. Reduction of ITS was found as 30% with sinusoidal loading tests as shown, whereas Type I increasing cyclic loading causes reduction of ITS by a maximum 35%.

For a clear understanding of the effect of rock fatigue on damage mechanisms, a comparison between static and dynamic cyclic loading tests is shown in Figure 6.5 by plotting both results on the same axes. Comparing these figures, diametral axial strain under both static and cyclic loading is bigger when compared with diametral lateral strain. However, the contribution of plastic strain to the diametral lateral strain plot under cyclic loading is much more than to that of the diametral lateral strain under monotonic loading. Moreover, diametral lateral plastic strain accumulation at the highest level as a result of loosening of the hysteresis loop during the final cycles up to failure. During cyclic loading, the diametral lateral deformation develops faster than the diametral axial deformation. This behaviour indicates the development of cracking in the specimen. This is the reason for dilatancy of the rock. The higher the stress level or the larger the cycle amplitude, the more obvious this behaviour is.

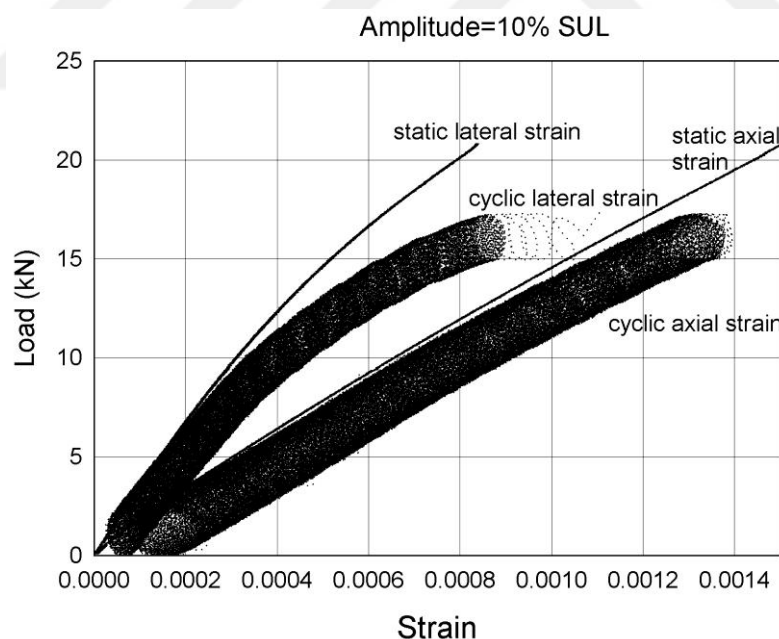


Figure 6.5 Comparison of load–strain curves of Brazilian disc specimens tested under static and Type I cyclic loading

Figure 6.6 shows the tested Brazilian disc specimens under static and cyclic loading. It is obvious that much more crushed rock material and debris was produced with cyclic loading tests compared with static tests. The mechanism of rock fatigue under cyclic loading induces many more

microcracks compared with the failure mechanism under static loading, as explained in the literature (Costin & Holcomb, 1981; Hadly, 1976). This experimental outcome is really interesting for investigating the effect of fatigue on damage mechanism of rocks. During both monotonic and cyclic tests, failure of specimens occurred very fast without gradual fracturing or any sign of predamage/cracking.

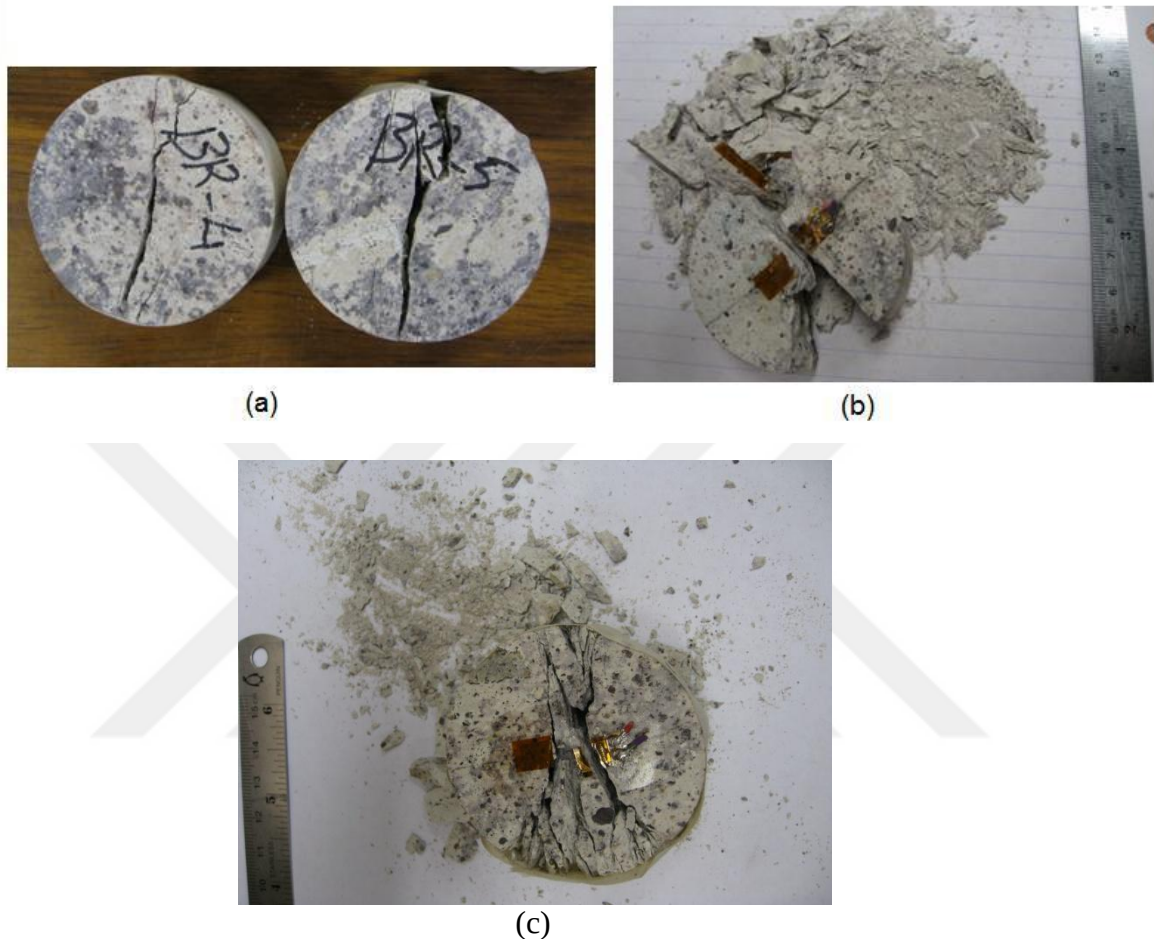


Figure 6.6 Tested Brazilian disc specimens under (a) static loading (b), (c) cyclic loading

Type II diametral compressive cyclic loading tests were performed on 12 CCNBD Brisbane tuff samples after the Type I series of cyclic tests. Results from the Type I cyclic tests appear to verify that with each amplitude value, the ITS is definitely reduced because of rock fatigue. This reduction was further explored through the second series of cyclic tests made by varying the valley values of each cycle at the same amplitude. Test results are given in Table 6.3. Comparing the results of Type I cyclic loading with Type II cyclic loading, the maximum ITS reduction was obtained with the smallest amplitude for Type II, whereas the maximum reduction of ITS was obtained with the biggest amplitude in Type I cyclic loading. It is believed this is because the smallest amplitude cyclic test has the highest mean stress level. This indicates the mean stress level is a more important factor in cyclic loading tests than loading cycle amplitude.

Table 6.3 Results of Type II cyclic loading tests performed on Brazilian disc specimens

Sample	Amplitude % SUL	Ultimate load (kN)	Number of cycles up to failure	Indirect tensile strength (ITS)	Reduction of ITS (%)
NST207-Rp1	10	14.6	735	6.87	36.4
NST207-Rp2	10	16.7	1106	7.86	27.3
NST207-Rp3	10	14.0	551	6.59	39
Average		15.1	797	7.10	34.2
NST207-Rp1	20	18.3	1049	8.61	20.3
NST207-Rp2	20	13.9	355	6.54	39.5
NST207-Rp3	20	17.2	955	8.09	25.1
Average		16.4	786	7.74	28.3
NST207-Rp1	30	13.8	241	6.49	40
NST207-Rp2	30	18.3	895	8.61	20.3
NST207-Rp3	30	15.3	435	7.07	35
Average		15.8	523	7.39	31.8
NST207-Rp1	40	16.1	384	7.58	30
NST207-Rp2	40	18.2	738	8.56	20.8
NST207-Rp3	40	17.2	513	8.07	25.3
Average		17.2	545	8.07	25.4

The Brazilian disc sample has two strain gauges attached to it on faces perpendicular to each other; lateral and axial to the force applied to the sample. Figure 6.7 shows the nomenclature of strain reading by the two strain gauges on a Brazilian disc specimen. The vertically placed strain gauge measures the vertical diametral strain, whereas the horizontally placed strain gauge measures the horizontal diametral strain.

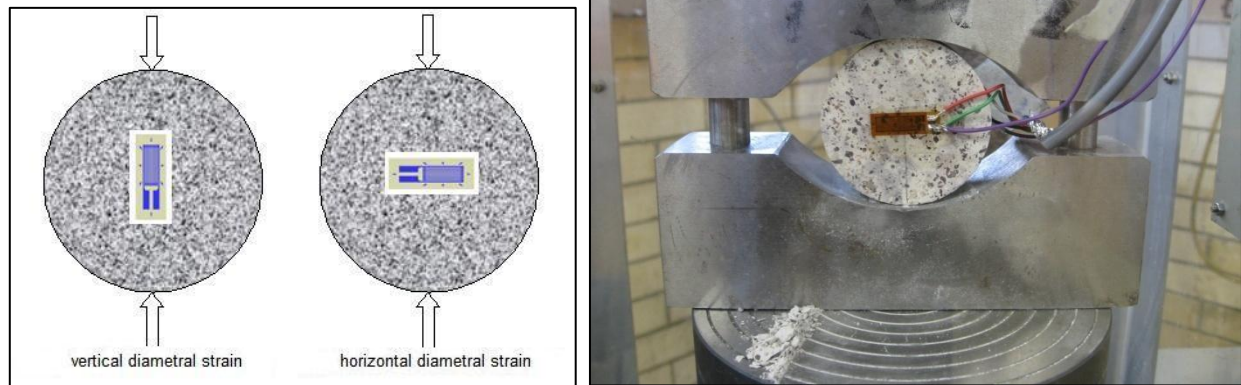


Figure 6.7 Vertical and horizontal diametral strain reading

Load–strain and strain–number of cycles plots are given in Figure 6.8 and Figure 6.9, respectively. Measurements of permanent strain show that both the axial and lateral strain curves increase with increasing number of cycles but at different rates. A slight deviation from linearity is evident only in the final stage of vertical diametral strain prior to failure, whereas remarkable deviation from linearity is evident in horizontal diametral strain (Figure 6.8a). The axial stress level where the axial strain departs from linearity marks the beginning of the onset of unstable crack growth, as defined by Bieniawski (1967). This generally occurs at axial stress level between 70 and 85% of the short-term peak strength of cylindrical samples. The dominant mechanism resulting in such an increase in axial strains is sliding along inclined surfaces, which are in the region characterised by the most significant structural changes to the sample, with the density of microcracks increasing by about sevenfold. As seen from Figure 6.8b, the fatigue damage of rock can be attributed to the continuous accumulation of stress-induced fracture damage. Examinations on the horizontal diametral strain changes reveal relatively high stiffness values when the load is first applied to the sample, up to 9 kN (the lateral stiffness term represents the change in the lateral strain rate with diametral compressive loading). The vertical diametral strain is almost double that of the horizontal diametral strain up to 70% of peak load (12 kN). Then the horizontal diametral strain rate increases very fast up to failure.

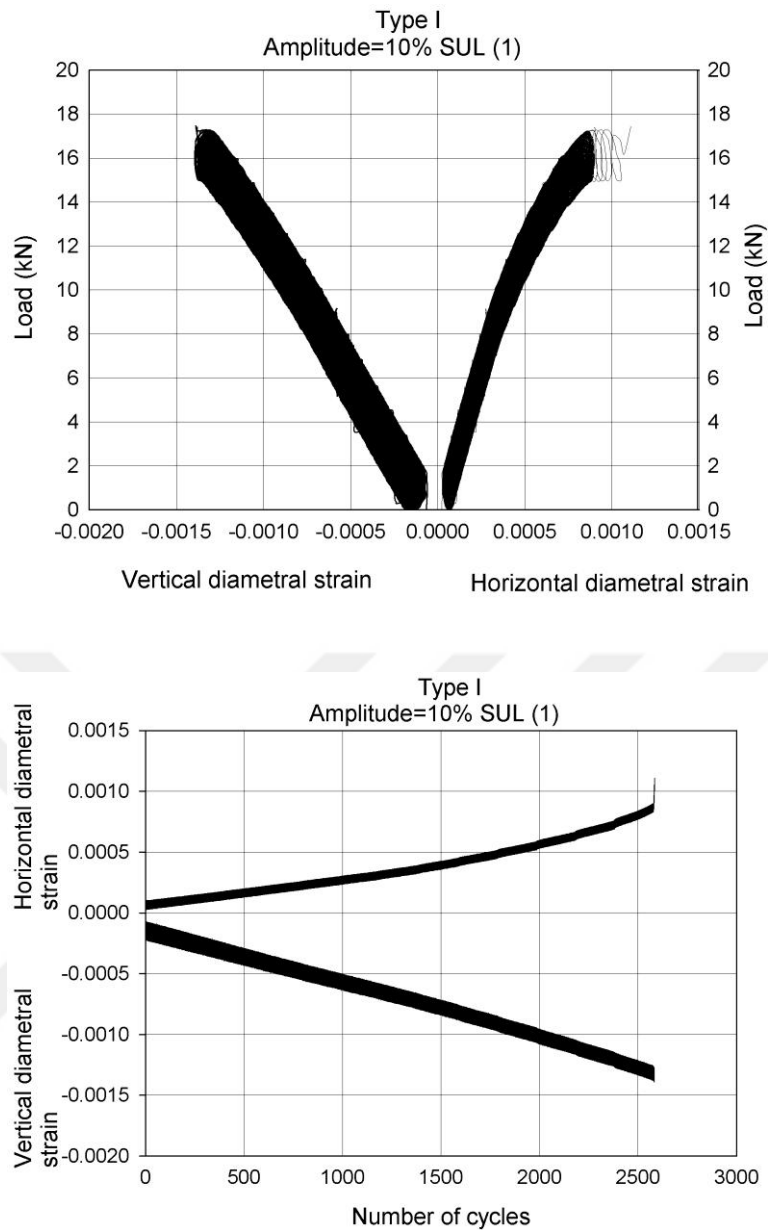


Figure 6.8 Type I diametral cyclic loading; amplitude = 2.3 kN (10% SUL)

The load–strain plots of Type II cyclic loading tests with the smallest amplitude (10% SUL) are shown in Figure 6.9. The loading cycles with the smallest amplitude in the Type II test were designed to have higher valley values than in the Type I cyclic tests. Thus, the first loading cycle starts from around 7 kN. At the beginning of the test, a specimen was loaded up to 7 kN with a specific monotonic loading rate suggested by ISRM (1981), then Type II cyclic loading was started. It is seen that the amount of diametral axial strain that occurred in a specimen under monotonic loading up to 7 kN is double that of diametral horizontal strain. This strain is the same amount of strain reached when the deviation from linearity starts in Type I cyclic loading (Figure 6.8a). It is possible to say that the damage in material obtained by Type II loading is slightly higher than in the Type I cyclic test. Comparing the vertical diametral strain that occurred in the material under Type I and Type II cyclic loading, the vertical diametral strain under Type II cyclic loading is slightly

higher than the relevant value for the Type I cyclic test. In contrast, there is not a significant difference in amount of accumulated horizontal strain in the disc specimens between Type I and Type II cyclic tests.

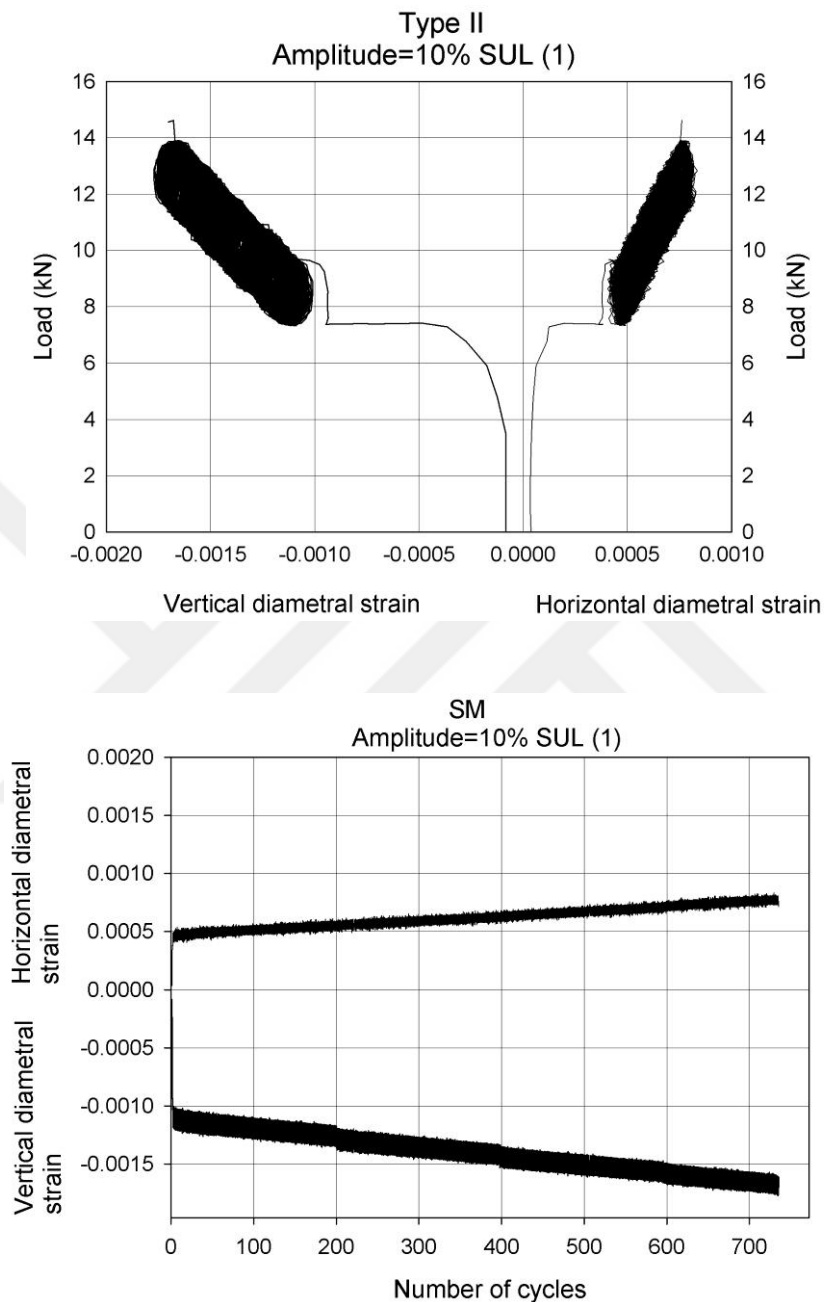


Figure 6.9 Type II diametral cyclic loading; amplitude = 2.3 kN (10% SUL)

An example of each Type I and Type II cyclic test repeated under various amplitudes is given in the following figures (Figures 6.10–6.15). The rest of the figures can be found in Appendices A. In general, there was not a big difference in amount of total strain produced in a specimen found between Type I and Type II cyclic loading. However, measurements of permanent strain damage show that both the axial and lateral strain damage curves increased with increasing damage

increments but at different rates in both types of cyclic tests. In general, the converse trends near the failure points result from the fact that lateral strain increase more rapidly than axial strain. It can be concluded that Type II cyclic loading produces the same amount of irreversible damage as Type I cyclic loading in a specimen, thus, equivalent damage in rock material is possible with fewer cycles and less energy with Type II cyclic loading compared with Type I cyclic loading.

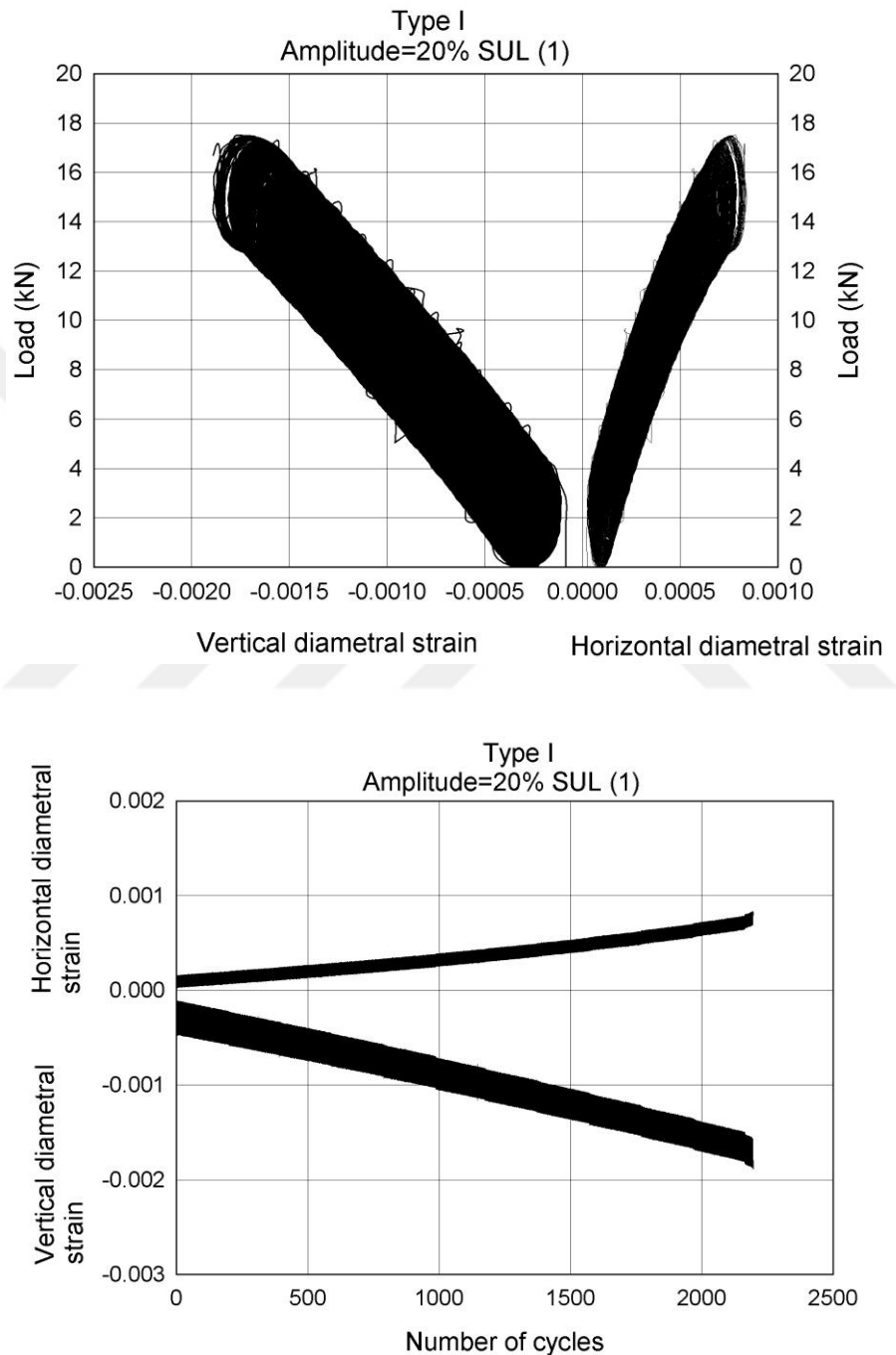


Figure 6.10 Type I diametral cyclic loading; amplitude = 4.6 kN (20% SUL)

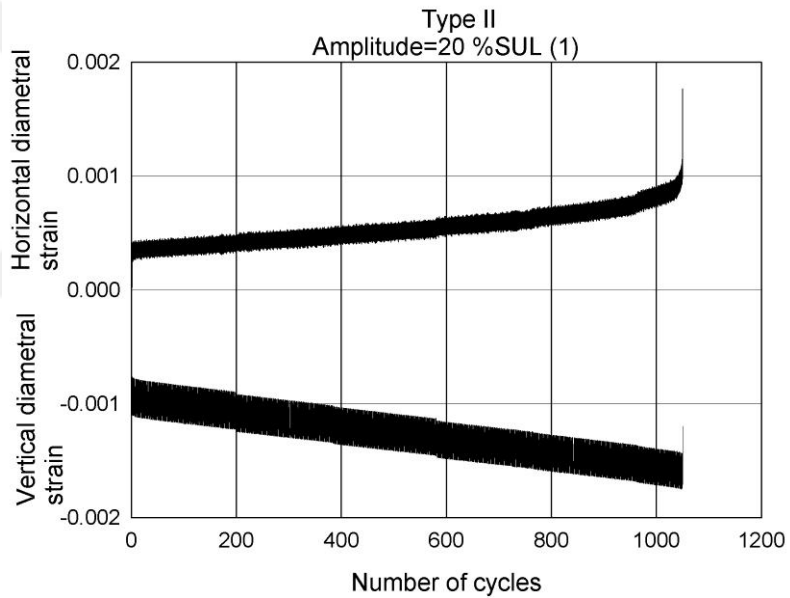
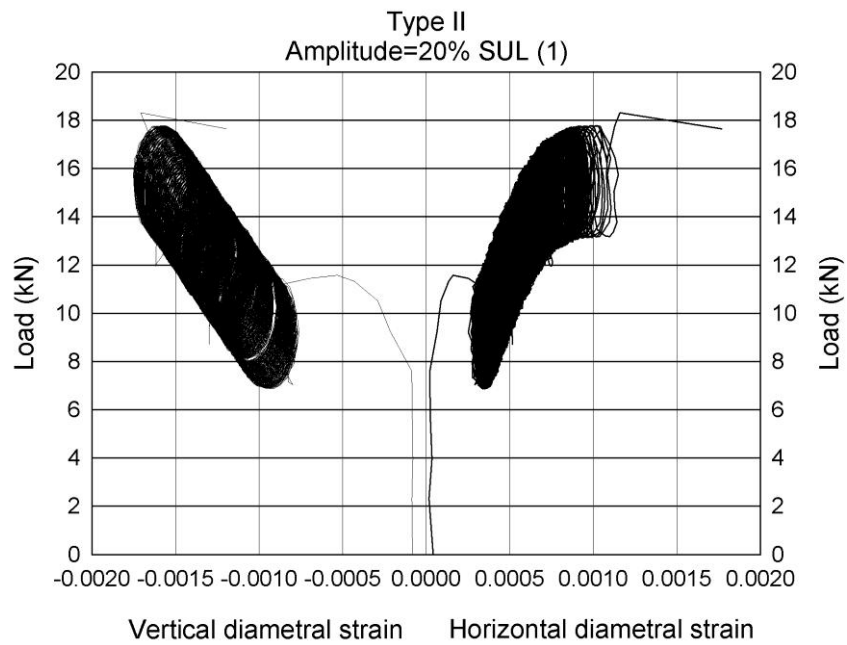


Figure 6.11 Type II diametral cyclic loading; amplitude = 4.6 kN (20% SUL)

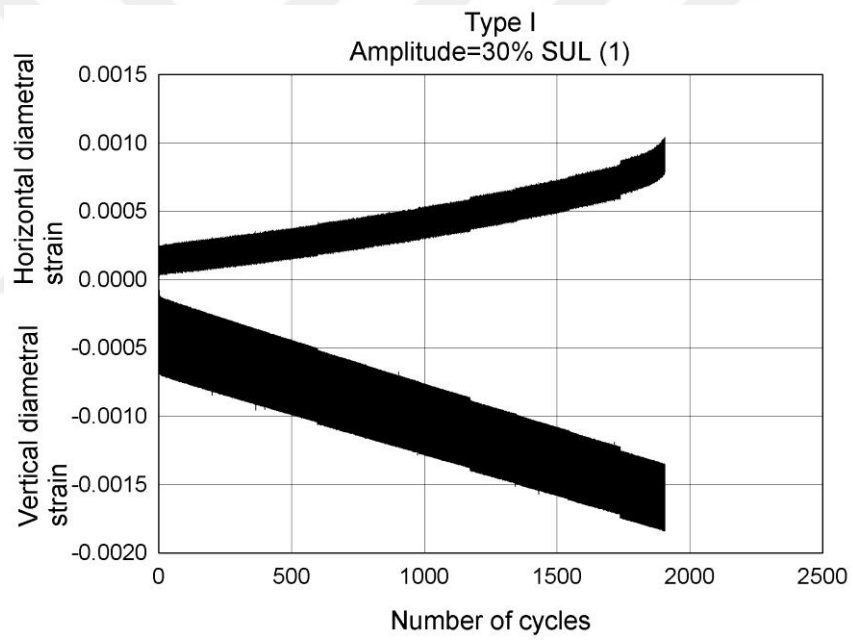
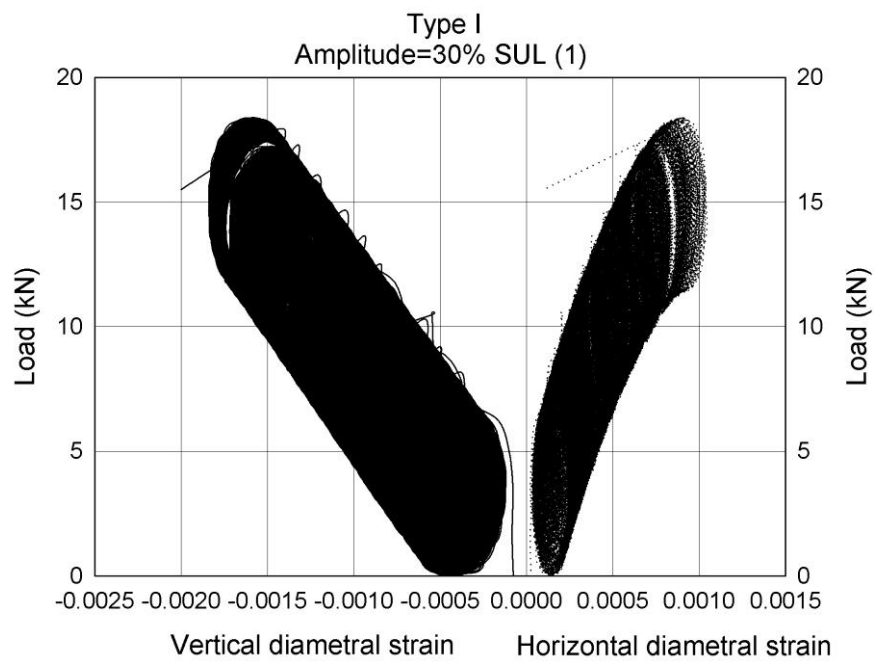


Figure 6.12 Type I diametral cyclic loading; amplitude = 6.9 kN (30% SUL)

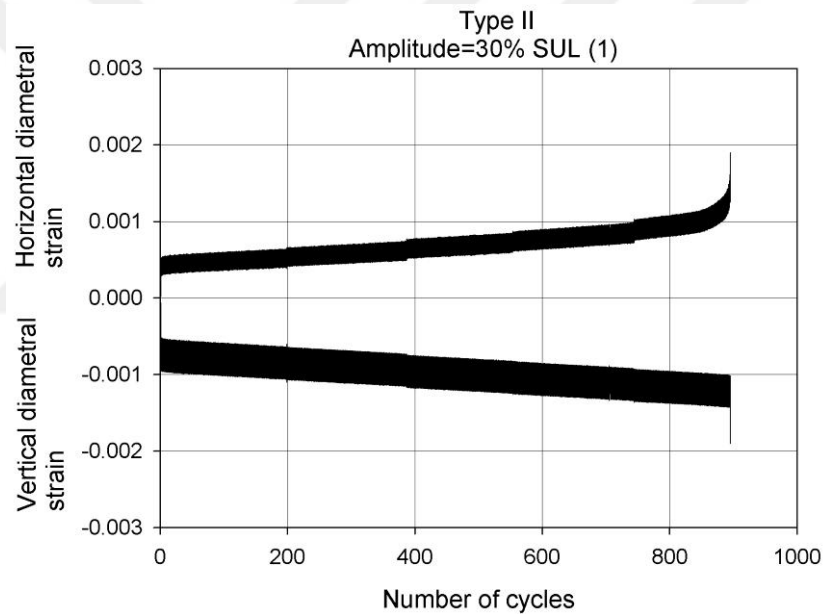
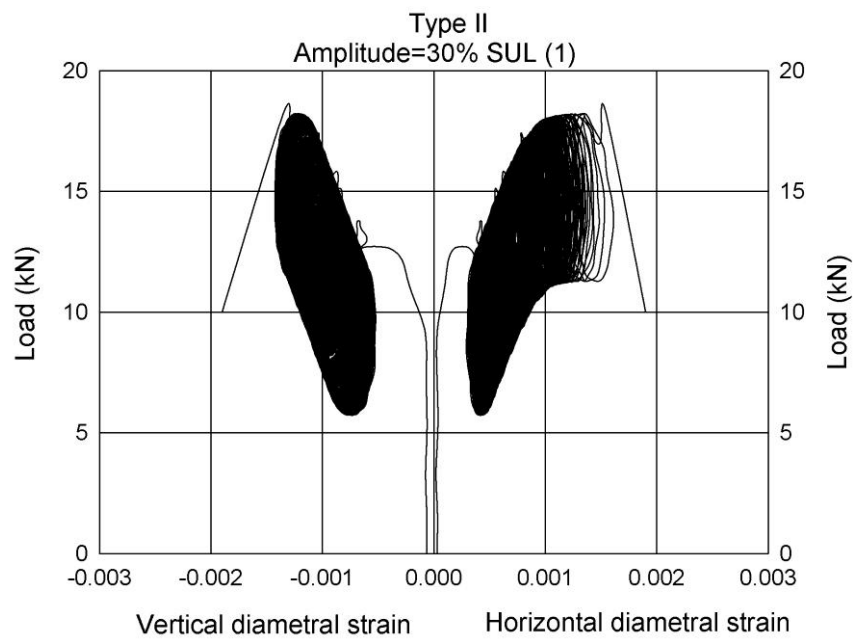


Figure 6.13 Type II diametral cyclic loading; amplitude = 6.9 kN (30% SUL)

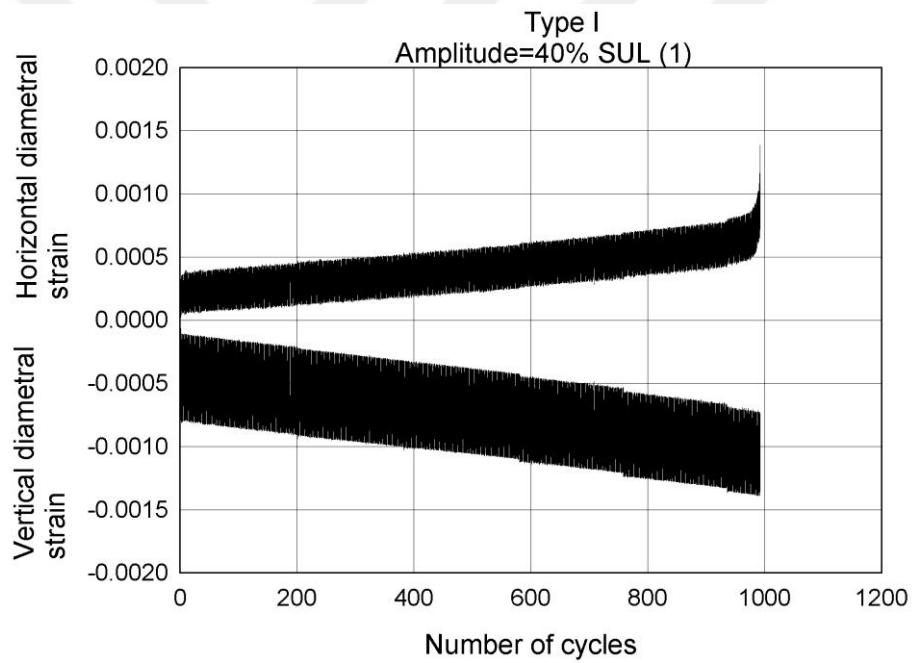
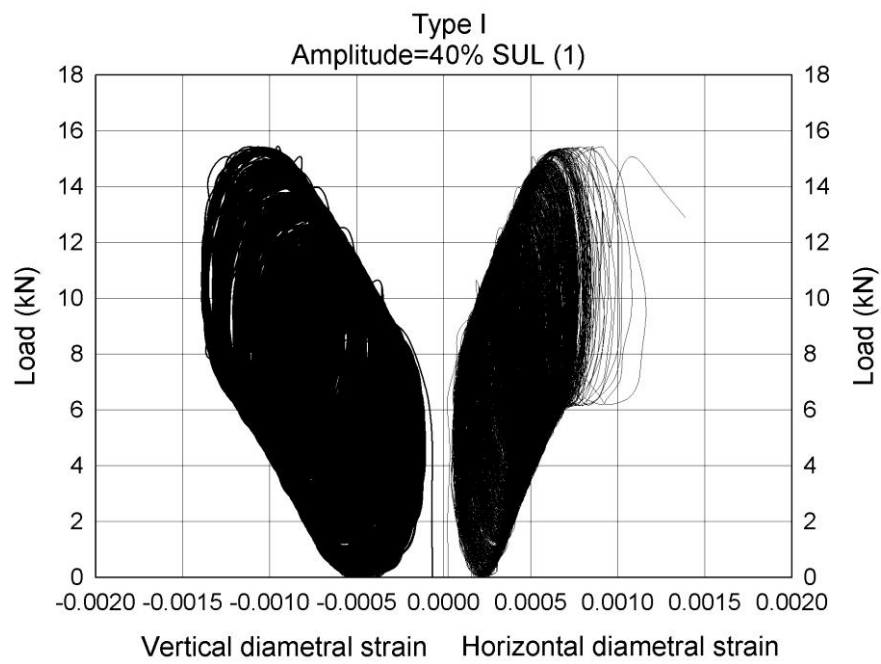


Figure 6.14 Type I diametral cyclic loading; amplitude = 9.2 kN (40% SUL)

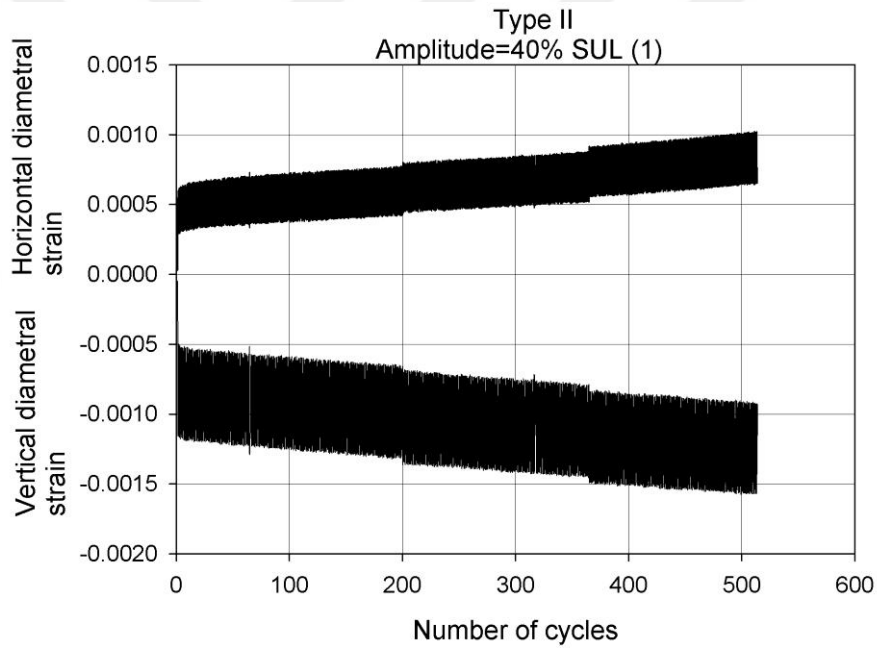
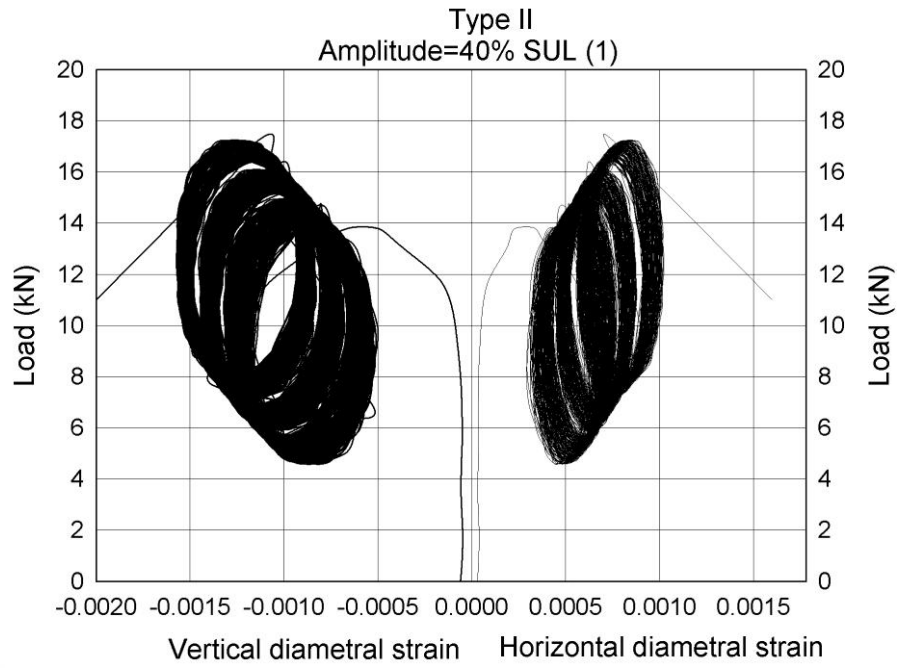


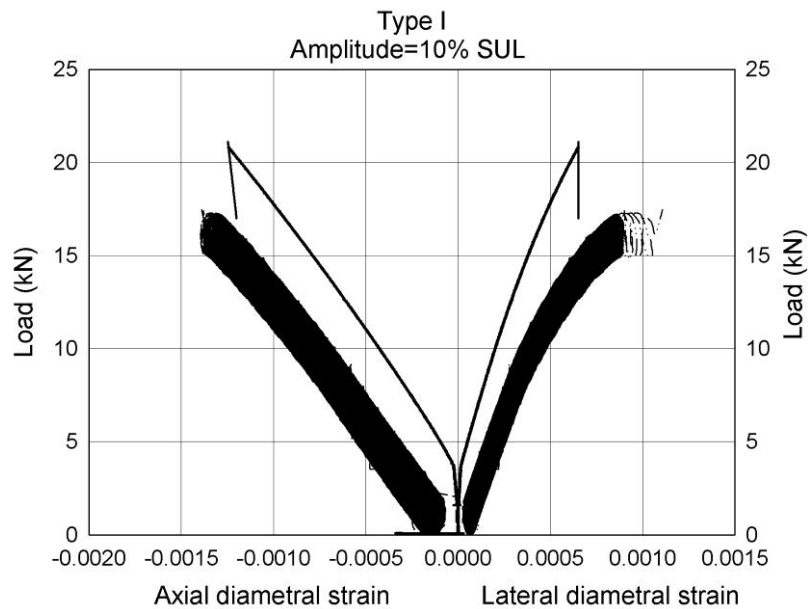
Figure 6.15 Type II diametral cyclic loading; amplitude = 9.2 kN (40% SUL)

6.3 Fatigue Damage Mechanism under Indirect Tensile Cyclic Loading

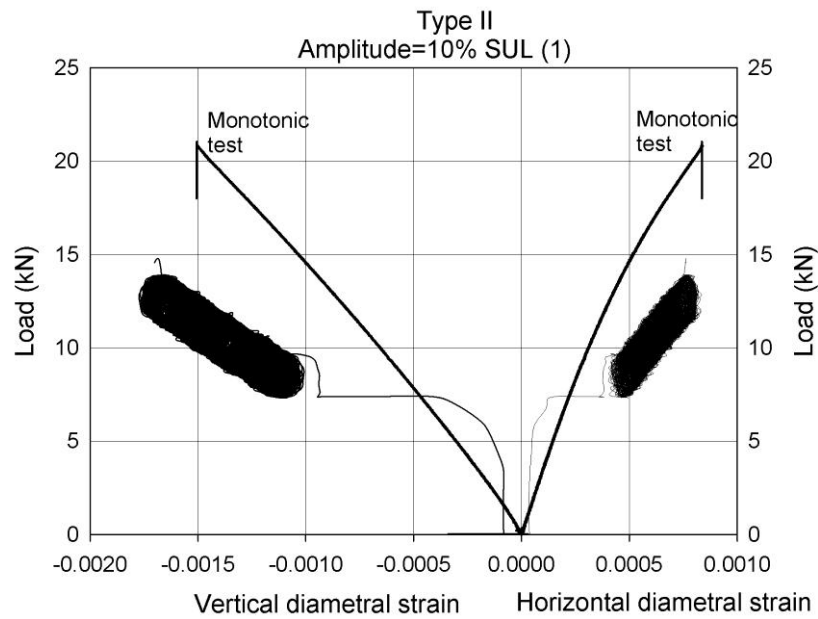
The damage mechanism caused by the loading rate–dependent dynamic tensile tests on rock material has been discussed by various researchers (Grady & Kip, 1980; Zhang *et al.*, 1999; Zhao & Li, 2001). It was generally concluded that Young's modulus and tensile strength of rocks obtained by the Brazil tests tend to increase with increasing loading rate. However, those studies mentioned above concern the mechanical properties of rock material under high loading rates (strain rates). In

contrast, for moderate to low loading rates, the proposed theoretical results were not found applicable (Nemat-Nasser & Deng, 1994; Zhao & Li, 2000). The result of all that discussion is that cyclic loading is not a rate-dependent type of loading. Therefore, the mechanism of tensile failure of rock material under cyclic loading needs to be studied.

For a clear understanding of the effect of rock fatigue on damage, a comparison between static and dynamic cyclic loading test results is shown in Figure 6.17, plotting both results to the same scale axis. The amplitude of the dynamic cyclic loading used in the tests was 2.3 kN (10% of the SUL). The ultimate load obtained from the Type I cyclic tests was reduced from the static value of 22 kN to 17.5 kN because of rock fatigue (Figure 6.17a). In contrast, the ultimate load obtained from the Type II cyclic tests was reduced from the static value of 22 kN to 14.2 kN because of rock fatigue (Figure 6.16b). The lateral deformation in the disc specimen shows loosening close to the failure point under Type I cyclic loading. This behaviour indicates the development of stress-induced cracking in the specimen. This is the reason for dilatancy of the rock. In general, the higher the stress level or the larger the cycle amplitude, the more obvious this behaviour is. In contrast, the damage mechanism in specimens failed under static or Type II cyclic loading is quite different (Figure 6.16b). In general, slightly more damage is produced in specimens with lower diametral compressive loads under cyclic loading compared with static tests.



(a)



(b)

Figure 6.16 Load–strain curves of Brazilian disc specimens under static and (a) Type I (b) Type II cyclic loading

Figure 6.17 shows load–strain plots for four cycles at the beginning of Type I increasing cyclic loading. Diametral axial plastic strain is higher than the diametral horizontal plastic strain during the first cycles. However, the amount of plastic strain in both vertical and horizontal directions after the first cycle is increasing by very small amounts compared with the first cycles. Continuous strain accumulation with each cycle up to failure was shown in prior strain–number of cycle plots. The last four cycles of the test are shown in Figure 6.18. The increasing rate of diametral axial plastic strain is very small whereas the produced diametral horizontal strain is very high, evidenced by the loosening of cycle loops (Figure 6.18b). The dilation of the cracks growing and coalescing along the diametral axial direction results in larger lateral strain compared with diametral axial strain. This shows that unstable crack propagation has started in the specimen. Unstable crack propagation continues to the point where the numerous microcracks due to fatigue coalesce into larger cracks and the rock can no longer support load cycles. It is believed that subcritical failure of rock, which is characterised by the development of a large number of microcracks, is possible even prior to the appearance of the main fatigue cracks in cyclically loaded specimens.

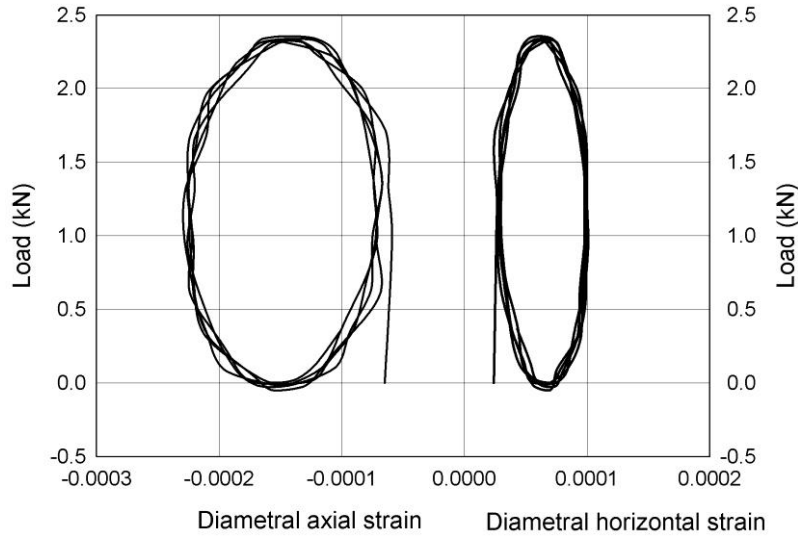


Figure 6.17 Load–strain plots of four cycles at the beginning of test

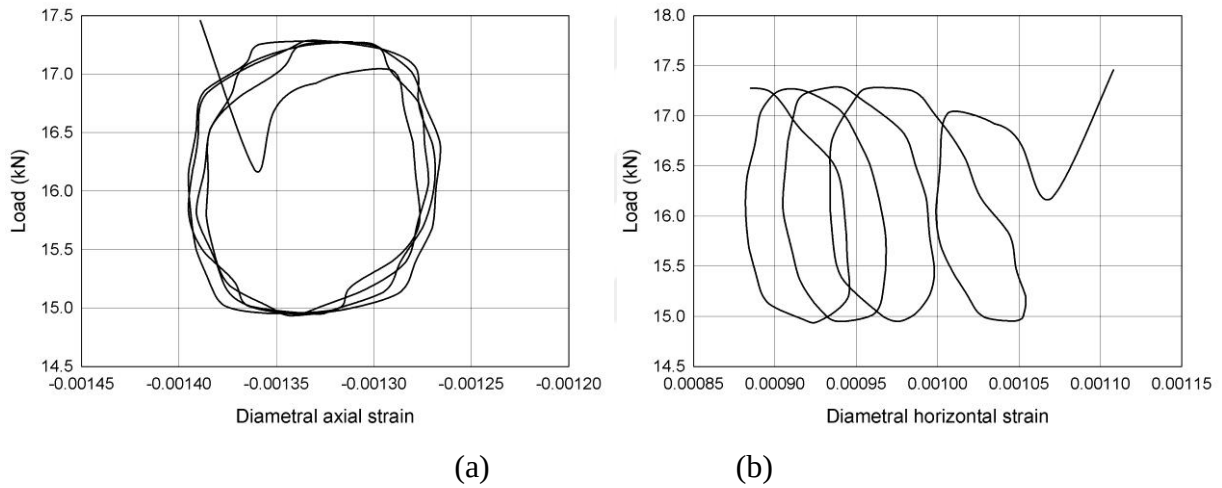


Figure 6.18 Load–strain plots of last four cycles up to failure

In order to examine the major characteristics of the damage process at the microscopic level, SEM imaging was used in this research. The failure surfaces of disc tested under both static and cyclic tests were examined by SEM. A JEOL JSM-6460 LA brand SEM device was used for tungsten low-vacuum analytical scanning electron microscopy. In this study, all rock fracture images were obtained under LV chamber pressures, i.e., 1–50 Pa (with adjustable pressure between 10 Pa and 270 Pa). This allows certain samples to be observed uncoated and reduces damage to the specimens caused by the effects of high vacuum. In addition, failure surfaces were examined with SEM directly without preparation of thin sections to avoid creating extra microcracks during preparation of a thin-section specimen.

Scanning electron micrographs of fracture surfaces of failed Brisbane tuff disc specimens tested under cyclic loading are shown in Figure 6.20. The petrographic details of Brisbane tuff are given in Chapter 3. According to the petrographic analysis results, quartz and feldspar minerals are infrequently packed in silica cryptocrystalline cement. In general, observation of the failed specimens with SEM showed that massive amounts of dust and small particles were produced in specimens tested under cyclic loading compared with monotonic loading. This outcome can be verified by examining the tested specimens shown in Figure 6.6 in macroscale. However, the damaged cement of Brisbane tuff and very small particles are shown in Figure 6.19 in microscale. Loosening and decohesion mechanisms seem very effective damage mechanisms in the cement. In contrast, the damage and fracturing mechanism in cement of Brisbane tuff disc specimens tested under monotonic loading is shown in Figure 6.20. Micro- and macroscale cleavage cracks are abundant and represent failure by splitting (pure opening mode) on the fracture surface of tested specimens under monotonic loading.

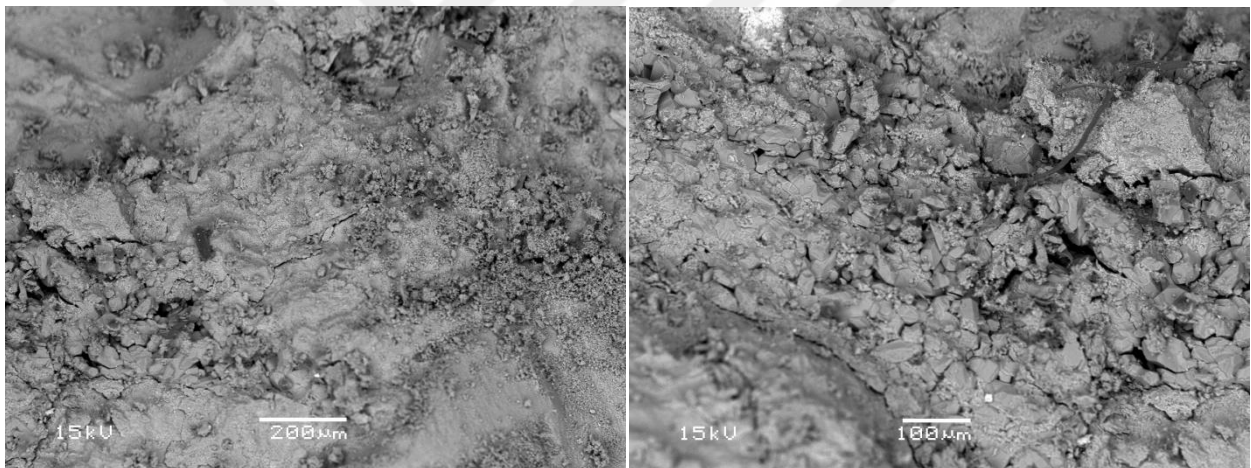
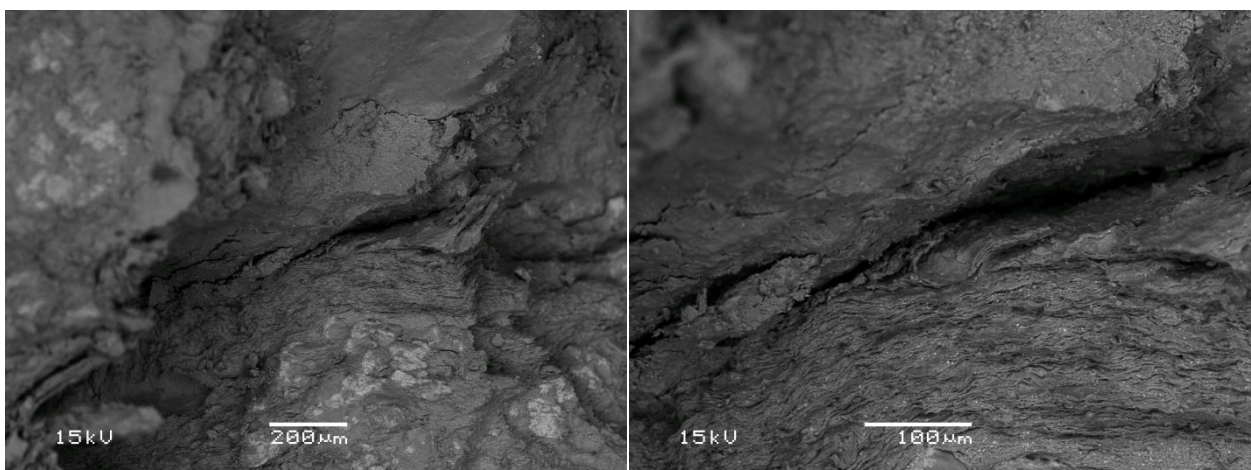


Figure 6.19 Damage and small particles in cement of failed Brisbane tuff disc specimen tested under cyclic loading



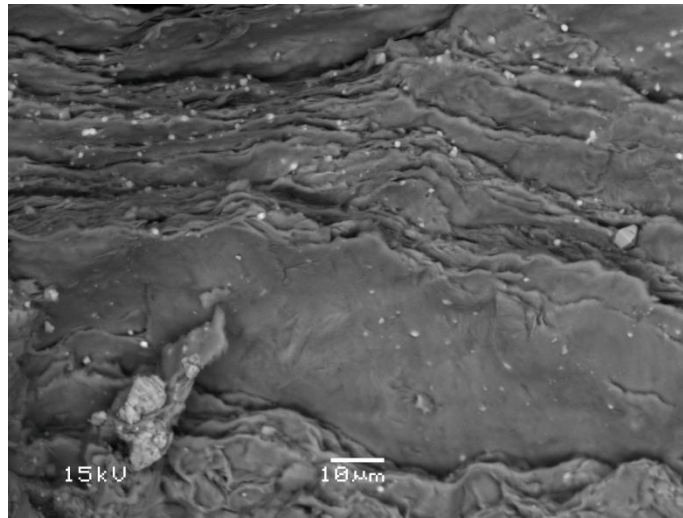


Figure 6.20 Damage and fracturing in cement of failed Brisbane tuff disc specimens tested under monotonic loading

Transgranular cracks run to or from a grain boundary and may transect an entire grain. Intracrystalline cracks begin and die out totally within one grain. Intercrystalline cracks cross over grain boundaries.

Most grains were crack free, and others relatively so, in fracture surfaces of tested specimens under cyclic loading, whereas almost all grains were highly cracked in the failure surfaces of tested specimen under monotonic loading. The cracking mechanism is brittle and cleavage cracking through minerals (Figures 6.21a and 6.21b). Because the quartz and feldspar minerals are placed freely in the cement of Brisbane tuff, a mismatch in elastic stiffness at the interface between feldspar or quartz grains that induces large tensile stress in the stiffer grain is not possible. This means that the cleavage mechanism due to high tensile stress produced in the centre of a disc specimen causes intergranular (intercrystalline) cracks in a disc specimen tested under monotonic loading (Figure 6.21a). The intergranular cracks did not die out in the grains and propagated through the cement in the specimen (Figure 6.21d).

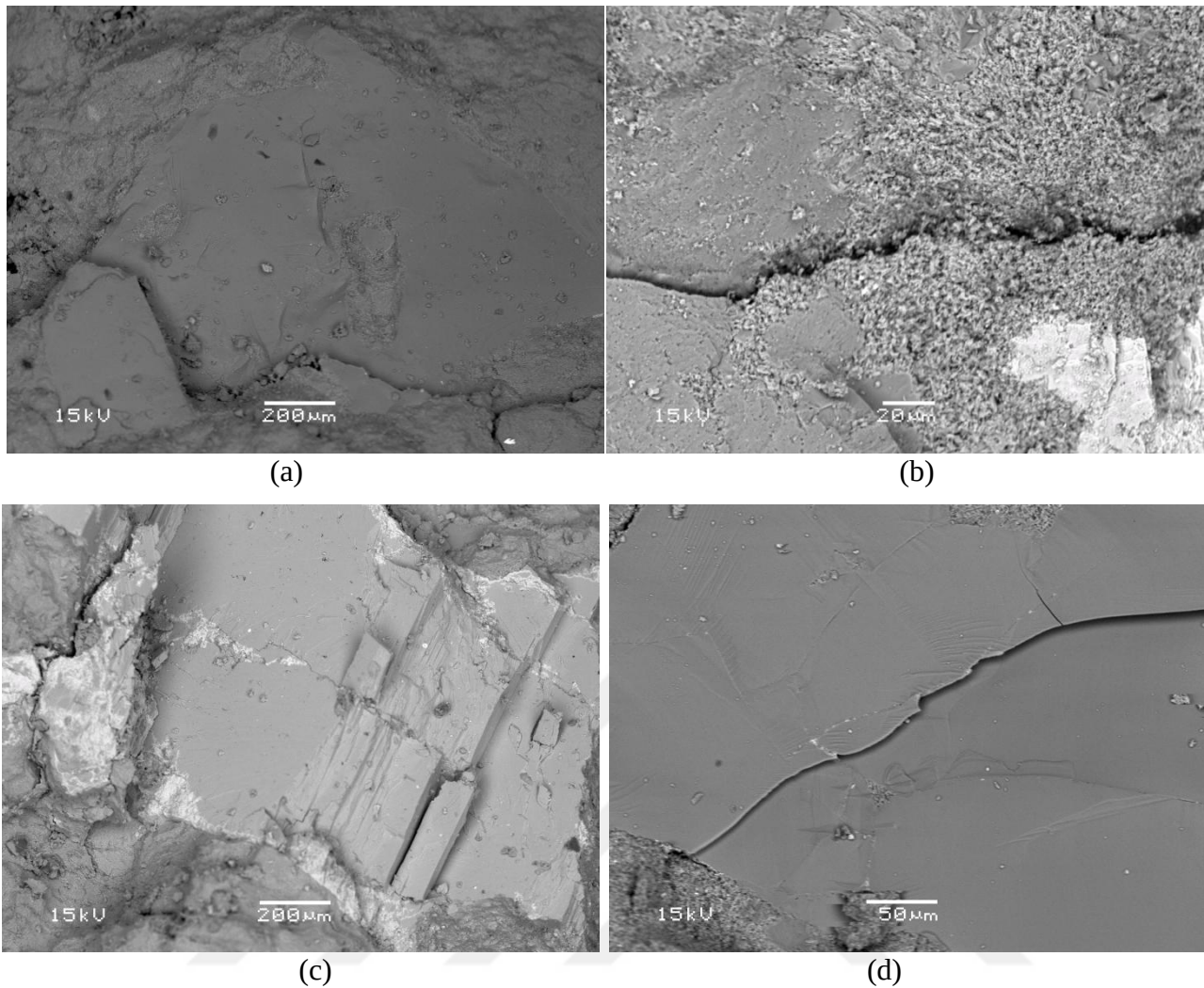


Figure 6.21 Typical transcrystalline cracks (a-b) and intracrystalline cracks (c-d) on failure surfaces of Brisbane tuff disc specimens tested under monotonic loading

Almost all grain boundaries are cracked along their entire length and some at small angles to the maximum stress direction on the failure surfaces of Brisbane tuff disc specimens tested under cyclic loading (Figures 6.22a and 6.22b). Pulled out loosened minerals shown in Figures 6.22c and 6.22d show general fatigue effects observed on failure surfaces of specimens in this thesis. In general, intercrystalline cracks were found as the principal cracking form in the fracture surfaces of cyclically fractured specimens. In contrast, it is already known that the fracturing mechanism in a disc specimen under diametral monotonic compressive loading is splitting (opening). However, the dust and small particles around loosened grains may indicate the fracturing mechanism of rock fatigue is produced by the combination of splitting (opening) due to stress-induced fatigue cracks and frictional sliding (shear) causing crushing at the grain boundaries during each loading and unloading cycle around loosened grains.

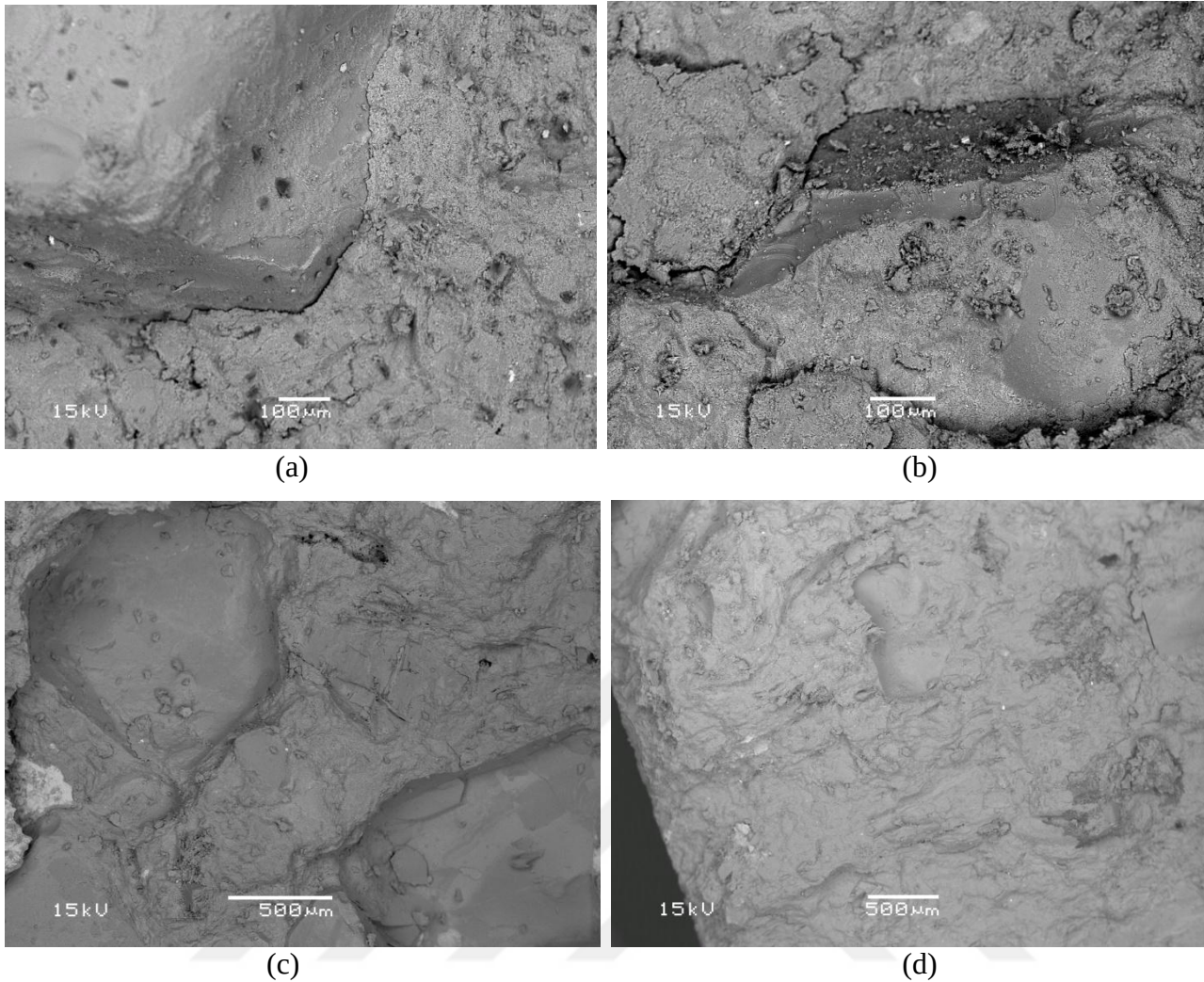


Figure 6.22 Scanning electron microscope images of the fractured surface of Brisbane tuff disc specimens (a) loosened grains and typical intercrystalline cracks around grains (b) pulled out grains from cement of tuff

Some amount of dust particles were observed around loosened grains by using high magnification of SEM (Figure 6.23). In general, point contacts at grain boundaries are regions of stress concentration. Transgranular cracks were found often emanating from these regions. However, those small particles around grains may be explained by indentation of a grain. An indentation pressure of stiffer grains on to the surface of weaker cement as an indenter tool is possible during each loading–unloading cycle. It is well known that some special cracks, such as radial cracks, propagate through the surface of a material during unloading of indenters (Tiryaki, 2008; Whittaker, 1992).

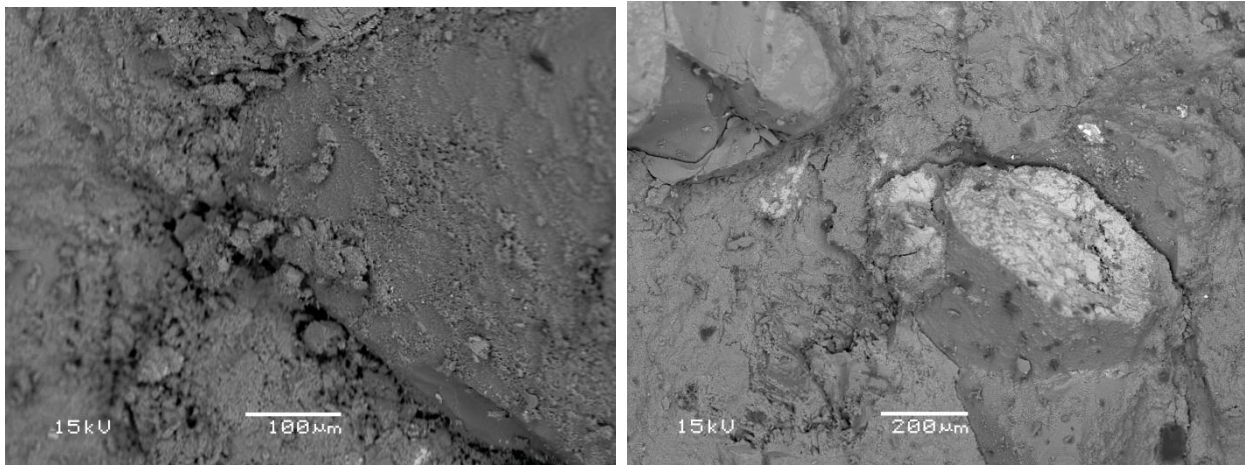


Figure 6.23 Scanning electron microscopic images of the fractured surface of Brisbane tuff disc specimens

From load–strain plots it was determined that fatigue fracturing is a continuous process consisting of continuing irreversible mechanical and physical deformation. According to the general findings from SEM images, the bonds between grains and cement are responsible for the fatigue damage of rock under cyclic loading.

6.4 Chapter Summary

The ITS response to cyclic loading was found to be different from that under static loading in terms of the lower ultimate load and different fracturing mechanism. A macroscale splitting crack with a sharp and rough surface was found along diametral compressive loading direction under monotonic loading, whereas an excessive amount of small particles and dust was produced under cyclic loading. In this research, various experimental observations of cyclic loading tests with varying amplitudes have indicated that increasing cyclic loading seems more effective on ITS degradation of rocks compared with sinusoidal compressive cyclic loading. Reduction of ITS was found to be 30% with sinusoidal loading tests, whereas Type I dynamic cyclic loading causes reduction of ITS to be a maximum 35%.

A big difference was not found between Type I and Type II cyclic loading in terms of reduction of ITS and fatigue damage mechanisms. However, Type II cyclic loading creates the same amount of irreversible strain with fewer cycles compared with Type I cyclic loading. An interesting outcome of the tests is the percentage of decrease in ultimate load increases with increasing load amplitudes. In contrast, the amount of reduction of ultimate load was found to be independent with varying

amplitudes. It is possible to conclude that Type II cyclic loading causes failure with fewer load cycles and even smaller loading amplitudes compared with Type I cyclic loading.

In addition to general outcomes, no postpeak behaviour was observed in load–strain plots of either monotonic and cyclic loading tests. Brazilian disc specimens failed violently during both monotonic and cyclic tests.

The SEM imaging results enable some of the qualitative features of the fatigue damage process in Brisbane tuff to be inferred. SEM investigations show that grain boundaries play an important role in rock fatigue. The largest cracks follow the grain boundaries and microcracks generally start to form at the grain boundaries or voids. Grain decohesion seems to be the source of large cracks. Also transgranular cracks are observed during rock fatigue. Mostly secondary cracks were seen on the fracture surfaces of a cyclically fractured specimen, but no such secondary cracks were found on a static fracture surface. The main characteristic is particle breakage under cyclic loading, which probably starts at contacts between particles and is accompanied by the production of very small fragments, probably due to frictional sliding within the weak matrix. It is believed that point contacts at grain boundaries are regions of stress concentration (i.e., indenters). Transgranular cracks may emanate from these regions, and intergranular cracks sometimes pass through the contact points. This stage can be correlated with a steady progression of damage and produces a general ‘loosening’ of the rock, which is a precursor to the formation of intergranular cracks.

CHAPTER 7

DEGRADATION OF FRACTURE TOUGHNESS OF ROCKS THROUGH CYCLIC LOADING

A principal objective of rock fatigue research in literature was to establish a relationship between the number of cycles and reduction of applied stress amplitude (S–N curve approach). A common finding of these research methods is that the repetition of loading cycles produces progressively more microcracks compared with monotonic tests. In contrast, relatively little attention has been given to investigating the damage mechanism of rock fatigue. A noteworthy outcome of this research is the observation of the effect of rock fatigue on fracture toughness of rocks.

One of the most fundamental parameters in fracture mechanics is fracture toughness, which describes the resistance of a material to crack propagation. Fracture toughness is an important material property that corresponds to the critical state of the stress intensity factor required for crack initiation and subsequent propagation. Assessment of resistance to crack propagation would be crucial in understanding the behaviour of structures involving brittle materials under cyclic loads.

Two different types of cyclic loading were used in this research; cyclic loading with constant mean level and constant amplitude, termed sinusoidal cyclic loading; and cyclic loading with increasing mean level and constant amplitude, termed increasing cyclic loading. Loading amplitudes are constant in both types of cyclic loading; however, the mean level of each cycle increases with a constant rate in increasing cyclic loading tests whereas the mean level of each cycle is constant in sinusoidal cyclic loading tests. Both kinds of cyclic loading tests were carried out on CCNBD rock specimens. Crack displacement was measured in terms of the CMOD. The tests were carried out on Brisbane tuff, which is the main host rock of Brisbane's first motorway tunnel, the CLEM7, from which core samples were obtained.

7.1 Sinusoidal Cyclic Tests

Sinusoidal cyclic loading test series were carried out to obtain an S–N curve, illustrative of the continuous weakening of rock with the increase in number of cycles (N) required for failing a specimen loaded to a certain upper peak stress (S) (Figure 7.1). In literature, the S–N curve concept has been used for fatigue research under uniaxial compressive loading with cylindrical rock samples

(Haimson, 1978; Haimson & Kim, 1971; Singh, 1989). However, this research is the first time obtaining an S—N curve for fracture toughness degradation because of fatigue under cyclic loads.

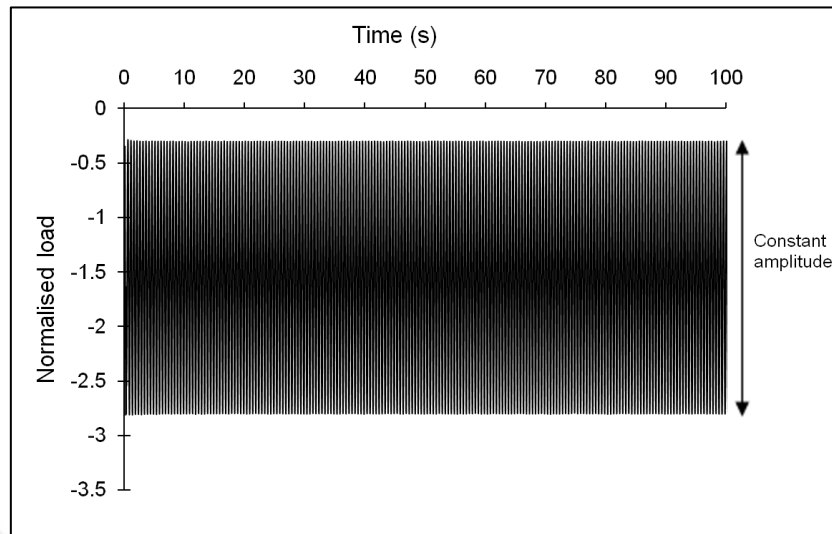


Figure 7.1 Sinusoidal cyclic loading

7.1.1 Determination of S—N curve for fatigue limit of CCNBD fracture toughness of Brisbane Tuff

The general procedure to obtain S—N is to reduce the maximum applied load from test to test until the fatigue life (no failure) of the specimen is reached. The loading frequency was 1 Hz during tests. The testing equipment was an Instron 6027 Rock Test System. The controller consists of hardware components and software applications that provide closed-loop control of servo-hydraulic test equipment. The Instron consists of three parts: a compression loading frame, an axial dynamic loading system and a data acquisition system. It is capable of conducting both uniaxial static and dynamic compression testing of rock specimens. The crack displacement was measured as CMOD across the crack mouth. A clip gauge for measuring the notch opening was attached to the knives. An Instron 2670 series crack opening displacement gauge was used to measure CMOD. The gauge length was 10 mm and maximum travel was 2 mm. The gauge met the requirements set out in British standard BS 5447 and American standard ASTM 399 70T.

Tests were carried out on Brisbane tuff, because it is a host rock of Brisbane's first motorway tunnel, CLEM7, from which core samples were obtained. The test specimens prepared were from standard Brazilian discs with a diameter of 52 mm and thickness of 26 mm (a diameter:thickness ratio of 0.5). The thickness, t , of the notches is 1.5 mm and the thickness of the samples, B , is 25

mm. The initial chevron notch crack length, a_0 , is 7.5–8 mm and the final chevron notch crack length is 36–37 mm.

The first important conclusion drawn from the cyclic tests is that the failure load used in calculations of fracture toughness is definitely weakened by repetitive loading. Results are given in Table 7.1.

Table 7.1 Results of sinusoidal cyclic loading

$S_{\max}$ (as % of static failure load)	$F_{\max}$ (failure load, kN)	Number of cycles to failure (N)	Percentage decrease in failure load
93	4.2	835	7
93	4.2	587	7
88	4	13	12
88	4	205	12
84	3.8	9	16
84	3.8	149	16
84	3.8	285	16
80	3.6	13000	20
80	3.6	27000	20
75	3.4	1761	25
75	3.4	4831	25
75	3.4	1153	25
75	3.4	24175	25
71	3.2	6250	29
71	3.2	50000*	29
71	3.2	50550*	29

*No failure

The S—N curve clearly shows that as the maximum applied diametral load decreases the life expectancy of a CCNBD specimen increases (Figure 7.2). Because the load range used in tests to determine fracture toughness is very small, the data around high amplitudes is clustered. However, the S—N curve shows that the ultimate load causing failure is reduced by 30% (from 4.5 kN to 3.2 kN) because of rock fatigue. Hence, it can be concluded that crack propagation resistance (fracture toughness) can be reduced by 29% through repetitive loading. This means that a crack can propagate under lower loads than the expected ultimate loads under repetitive loading.

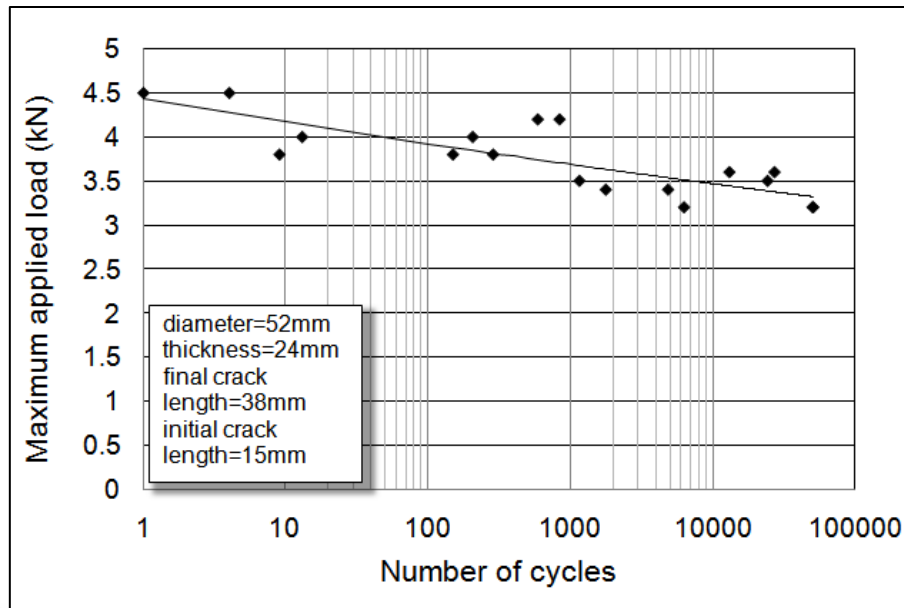


Figure 7.2 S—N curve for CCNBD specimens

7.2 Effect of Increasing Cyclic Loading on Mode I Fracture Toughness

The cyclic loading used in one of the two different series of tests was ramp type cyclic compressive loading with increasing mean level, termed dynamic cyclic loading. The loading frequency was 1 Hz for all tests. The amplitude is expressed as an absolute ($\pm$ value) equal to the total range. The illustration of dynamic cyclic loading is given in Figure 7.3. Four different amplitudes were chosen to investigate the effect of fatigue on the fracture toughness of rocks: 0.45 kN at 10% Static Ultimate Load (SUL), 0.9 kN (20% SUL), 1.35 kN (30% SUL) and 1.8 kN (40% SUL).

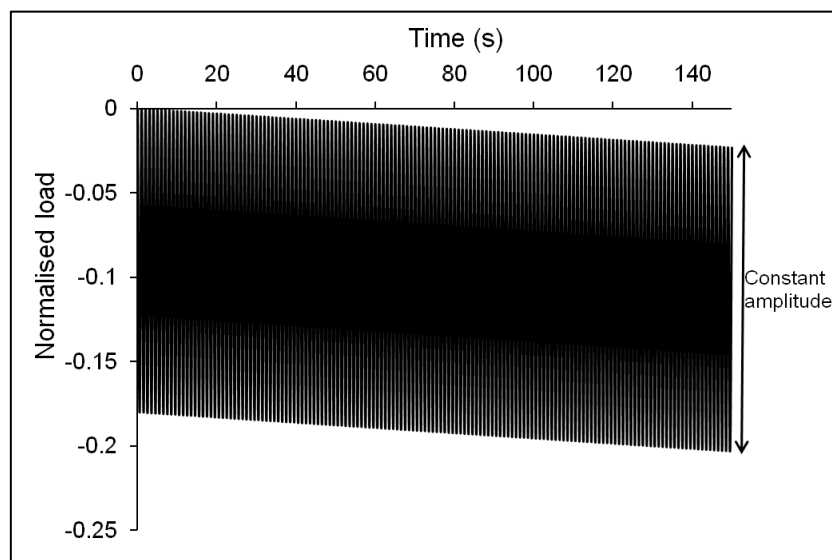
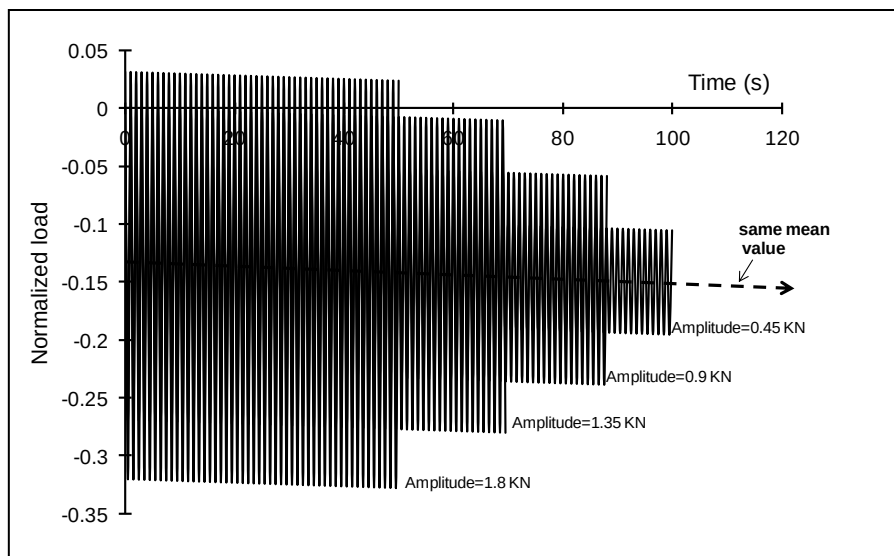


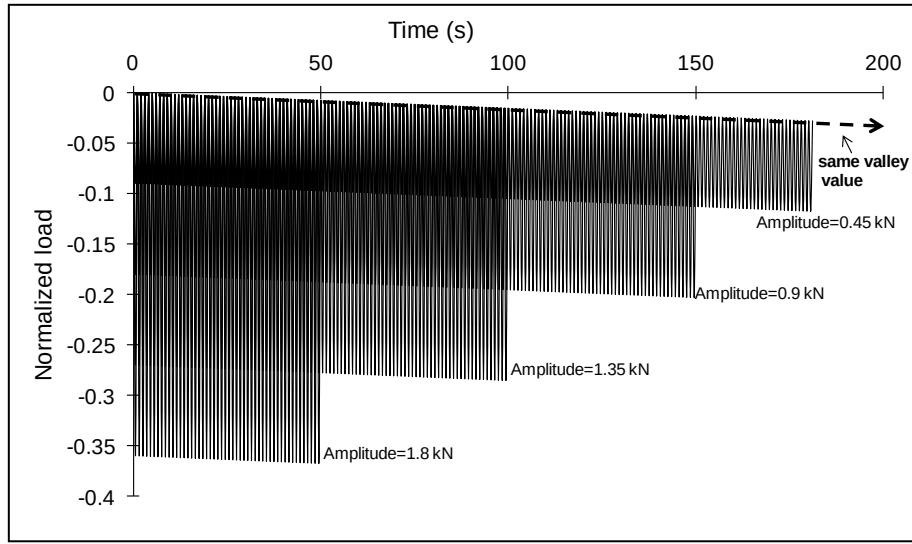
Figure 7.3 Load–number of cycles graph illustrating dynamic cyclic loading

In general, obtaining an S—N curve is quite a time-consuming process. The approximate time to conduct a test with low amplitudes took two days in this research. Thus, data was recorded at peak and valley points for force and displacement readings instead of acquiring the data all through one complete sinusoidal cycle. Thus, it was decided to conduct a dynamic cyclic loading test series to get a proper data record in a shorter time. Because dynamic cyclic tests are less time consuming and produce a full data record through a complete cycle, CMOD, force, displacement and time data was able to be recorded for all amplitude values up to failure.

Two types of dynamic cyclic loading test series were performed on Brisbane tuff specimens: (1) dynamic load cycles with the same increasing valley values (termed Type I) (Figure 7.4a) and (2) dynamic load cycles with the same increasing mean values (termed Type II) (Figure 7.4b). Amplitudes are same in both cyclic loading types. The main purpose of performing those two types of cyclic loading was to find the most damaging cyclic loading type using the same amplitudes. It is known that the act of applying a load provides energy for the crack initiation and propagation process in a material. By removing the load, the energy supply driving crack propagation is discontinued and the remaining excess energy in the system dissipates through the crack propagation processes. Hence, the point where the unloading of a cycle stops becomes important. Therefore, testing followed a procedure to investigate the effect of amplitude by using a setup with the same amplitude and the same unloading stop point (valley) and the same amplitude with varying unloading stop points (valley). Moreover, although amplitudes are same in both Type I and Type II cyclic loading, the mean levels of load cycles used in Type II cyclic loading are higher than the mean levels of load cycles used in Type I cyclic loading. Thus, more damage was expected with Type II cyclic loading when experiments were planned at the beginning of the research.



(a)



(b)

Figure 7.4 Dynamic loading cycles (a) with same increasing mean values (b) with same increasing valley values

7.2.1 Effect of Cyclic Loading with Increasing Valley Values on Mode I Fracture Toughness

The series of diametral compressive dynamic cyclic loading tests was performed on 12 CCNBD Brisbane tuff samples. The analyses of these tests were focused on the reduction of mode I fracture toughness (K_{IC}) under dynamic cyclic loading with different amplitudes. Fracture toughness values of Brisbane tuff under static loading were calculated as $1.12\text{--}1.5 \text{ MPa}\sqrt{\text{m}}$ using ISRM (1978) suggested methods as given in previous chapters. The fracture toughness reduction because of cyclic loading is given in Table 7.2 by comparing with static fracture toughness values. The main purpose of this comparison is to show the clear reduction of ultimate failure load that resulted in the reduction of mode I stress intensity factor because of rock fatigue. We try to show that crack propagation causing failure is possible with lower stress intensity values (K_I) at the crack tip than the critical stress intensity value (K_{IC}). This result goes against the classical theory, which predicts that there will be no crack growth as long as $K_I < K_{IC}$.

According to the obtained results, the maximum reduction, 46% of static K_{IC} , was obtained with the highest amplitude, 1.8 kN (40% SUL) of cyclic loading. The mode I fracture toughness value of Brisbane tuff (NST50) is reduced from $1.12 \text{ MPa}\sqrt{\text{m}}$ to $0.61 \text{ MPa}\sqrt{\text{m}}$ with the highest amplitude under increasing cyclic loading. This reduction is an impressive finding for investigation of the effect of cyclic loading on fracture resistance of cracks in rocks.

Table 7.2 Results of Type I cyclic loading tests on CCNBD specimens

Sample	Amplitude % SUL	Ultimate load (kN)	Number of cycles up to failure	Mode-I Stress Intensity Factor (K_I)* MPa√m (Shetty, 1985)	Fracture toughness (K_{IC})** MPa√m	Reduction of K_{IC} (%)
CCNBD-Rp1	10	2.39	2453	0.62	0.72	35
CCNBD-Rp2	10	1.95	1829	0.50	0.60	45
CCNBD-Rp3	10	2.69	2261	0.69	0.83	26
Average		2.34	2181	0.61	0.72	35
CCNBD-Rp1	20	2.16	1553	0.55	0.67	41
CCNBD-Rp2	20	2.10	1501	0.53	0.65	42
CCNBD-Rp3	20	2.24	1630	0.57	0.70	38
Average		2.17	1561	0.55	0.67	40
CCNBD-Rp1	30	2.40	1218	0.62	0.74	34
CCNBD-Rp2	30	2.5	1702	0.64	0.77	32
CCNBD-Rp3	30	2.42	1501	0.62	0.75	33
Average		2.44	1473	0.63	0.75	33
CCNBD-Rp1	40	1.86	319	0.48	0.57	50
CCNBD-Rp2	40	2.17	446	0.55	0.67	41
CCNBD-Rp3	40	1.95	136	0.50	0.60	47
Average		1.99	300	0.51	0.61	46

* α is assumed at 0.5, ** Y_{min}^* is assumed at 1.21

To date, various experimental observations of cyclic loading tests have indicated that dynamic cyclic loading seems more effective on fracture toughness degradation of rocks than sinusoidal compressive cyclic loading. The reduction of fracture toughness was found as 29% under sinusoidal loading tests as shown, whereas Type I dynamic cyclic loading causes reduction of fracture toughness at a maximum of 46%.

For a clear understanding of the effect of rock fatigue on damage mechanism, a comparison between static and dynamic cyclic loading tests is shown in Figure 7.5 by plotting both results on the same axes. The amplitude of the dynamic cyclic loading used in the test was 0.45 kN; 10% of Static Sltimate Load (SUL). As seen from Figure 7.5, both failure load values and damage mechanisms are quite different under static and dynamic cyclic loading. The failure load obtained

from the average of two static tests was reduced from 4.2 kN to 2.1 kN because of rock fatigue. In both test types, stable and unstable crack propagation stages are clear, however, the resistance of crack propagation to cyclic loading with accumulation of plastic deformation (0.8 mm) is much more compared with the relative value (0.025 mm) under static loading before failure (Figure 7.5). This behaviour shows that development of a large number of microcracks causing accumulation of irreversible deformation is observed even prior to the appearance of main cracks in a loaded specimen. This phenomenon is similar to the subcritical failure of geomaterials commonly known as subcritical crack propagation. This mechanism is discussed in detail in the following sections.

A load–CMOD plot further reveals that there is a clear tensile softening behaviour with dynamic cyclic loading graph. Because of the different postpeak behaviour, the behaviour of the damage zone at the tip of the chevron notch inside the sample is impressively different. This may help to explain the fatigue mechanism by using the FPZ concept in front of the cracks. In contrast, any tensile softening behaviour postpeak was not observed with static loading tests given in the previous section (Figure 7.5). These conclusions are quite impressive and helpful to explain rock fatigue mechanism in rocks.

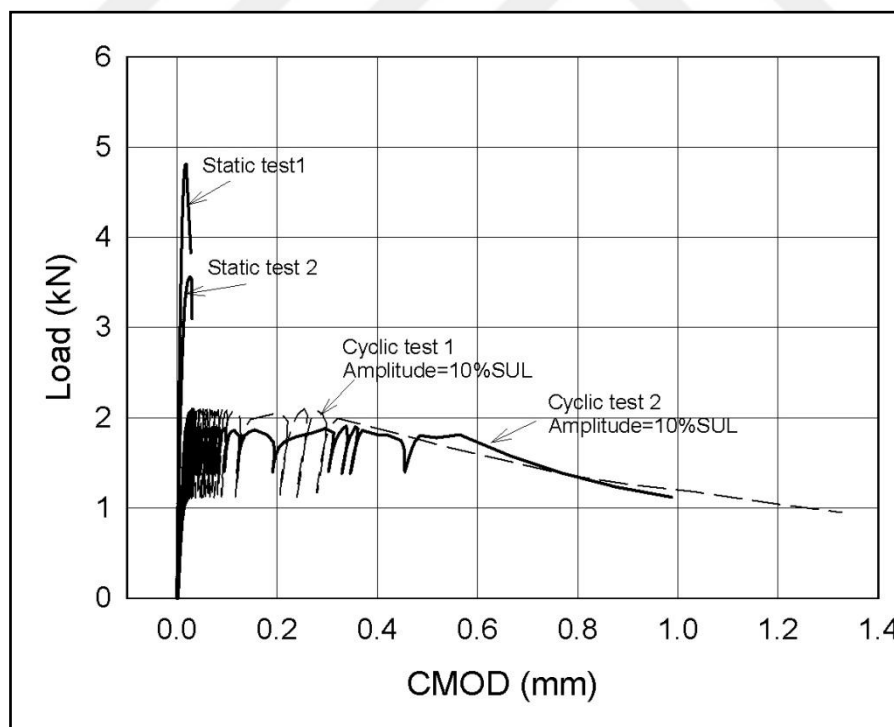


Figure 7.5 Comparison of load–CMOD curves of CCNBD specimens tested under static and Type I cyclic loading

Another important observation was made by examining the crack surfaces of failed specimens after dynamic cyclic loading tests. There is a clear crushed region including small particles and dust in

front of the chevron tip, as shown in Figure 7.6b. However, any rock chips (small particles) were not observed at the crack surfaces of failed specimens under static loading, as seen in Figure 7.6a. High micro- and macrocrack density in front of the chevron tip shows a gradual separation of crack surfaces along a preferential path set up by microcracking. This damaged zone in front of the chevron tip is called the FPZ. The mechanism of rock fatigue under cyclic loading has been explained in the literature with many more microcracks induced compared with failure mechanism under static loading (Costin and Holcomb, 1981; Hadly, 1976). Hence, observation of the FPZ zone in front of the chevron notch in CCNBD specimens would be a good explanation to investigate fatigue mechanism.

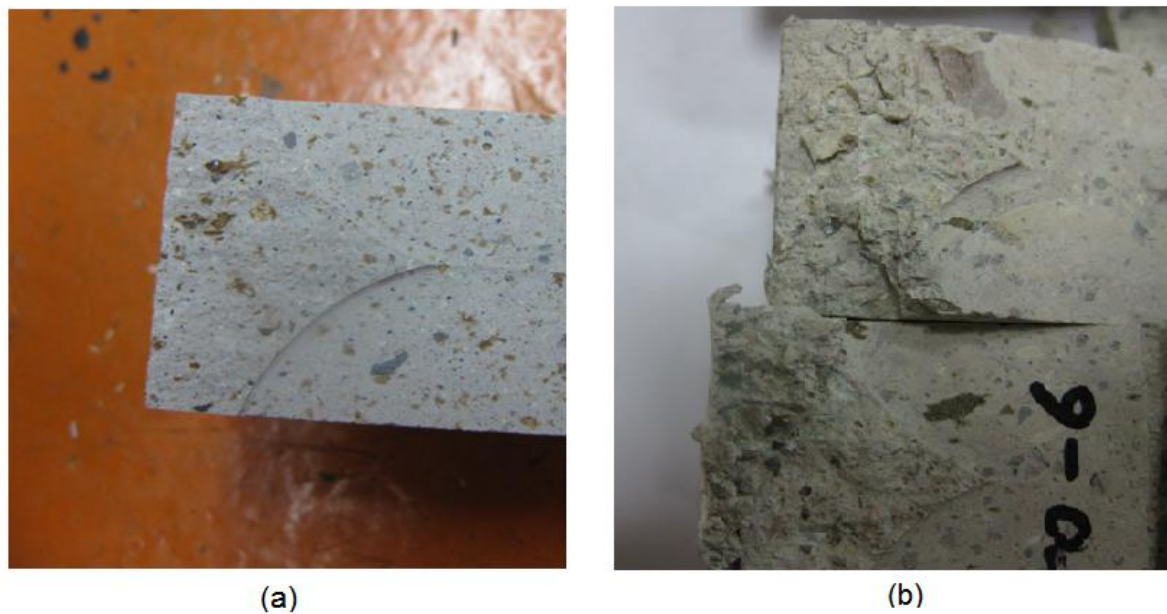


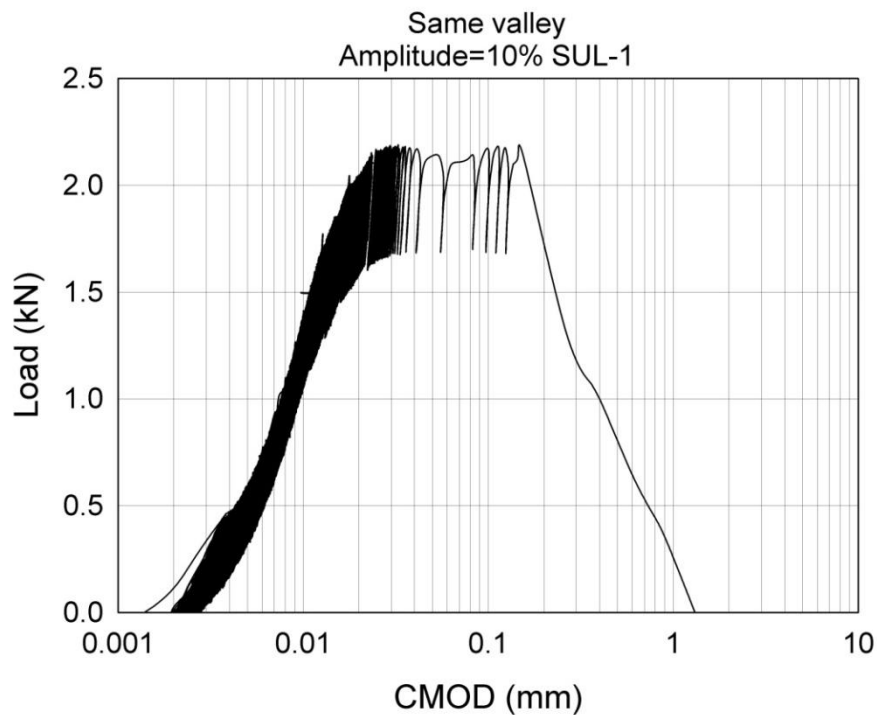
Figure 7.6 Failed specimens and damaged zone in front of the chevron tip (a) under static loading (b) under cyclic loading

Figures 7.7–7.10 show the load–CMOD plots with different amplitudes of dynamic cyclic loading tests. To avoid bounding and moving of the samples, a 0.5 kN minimum compressive load was applied at the beginning in all tests. The loading–unloading frequency was 1 Hz. Because the plastic deformation is very high compared with elastic deformation in the plots, the CMOD axis was plotted as a log scale. The general finding from these plots is that considerable stable crack extension(s) with plastic strain accumulation is possible under indirect tensile cyclic loading even with various amplitude values. In addition to that general finding, a clear tensile softening took place for all postpeak behaviours in load–CMOD plots after an accumulated plastic deformation in the sample. However, it was not possible to get tensile softening or postpeak behaviour after a

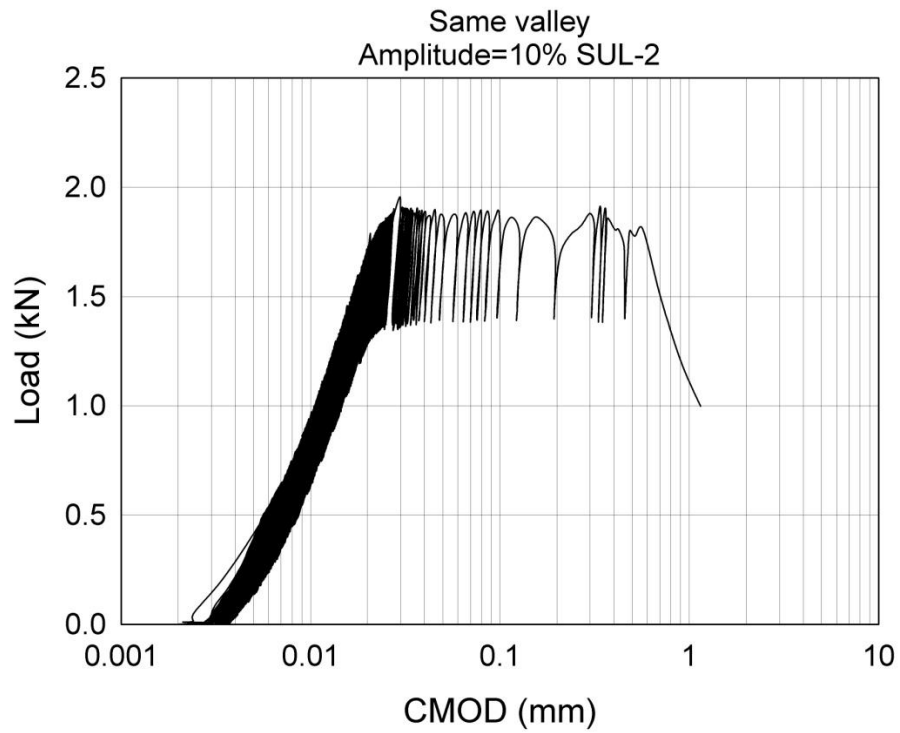
plastic deformation with static loading tests. A detailed discussion of this behaviour is given in the following sections.

It can be seen from the graphs that failure took place between 0.3 mm and 3 mm CMOD values after the start of the softening region of each plot. It is interesting to note that the obtained maximum CMOD value before failure from static loading tests was around 0.03 mm. This remarkable difference shows that a dominant dilation region occurs in front of the chevron notched crack tip under dynamic cyclic loading compared with relative values under static loading.

The load–CMOD plots further reveal that no elastic deformation occurred before plastic deformation starts with the highest amplitude cyclic loading tests (Figure 7.10). It is clear that the amount of elastic deformation decreases with increasing amplitude of dynamic cyclic loading. In contrast, the maximum amplitude cyclic loading tests were found to be the most damaging test with a reduction of 46% in fracture toughness in all cyclic tests (Table 7.2).

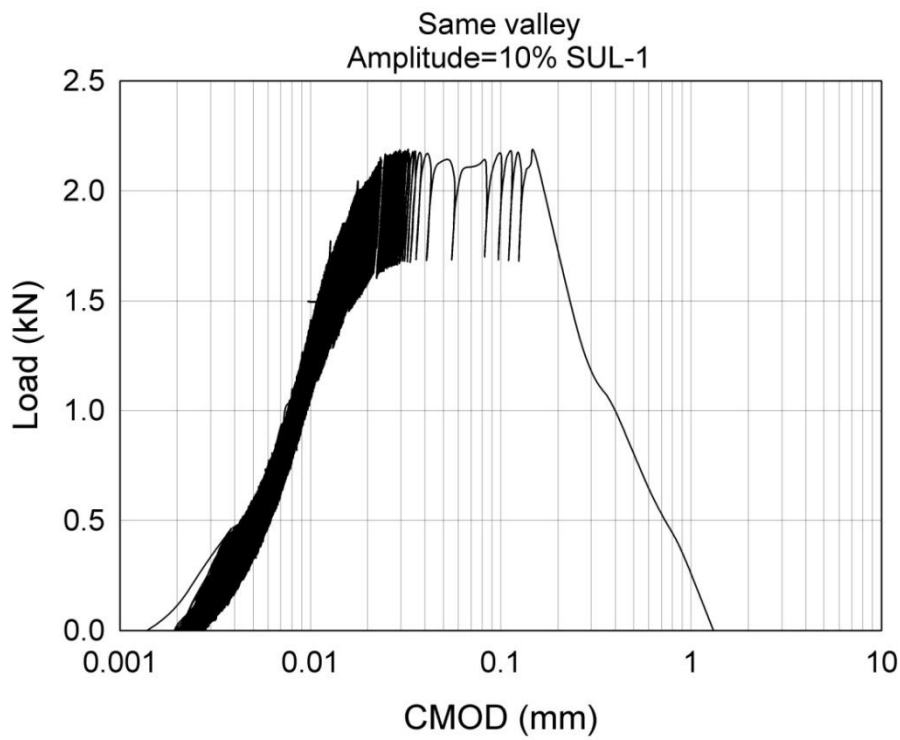


(a)

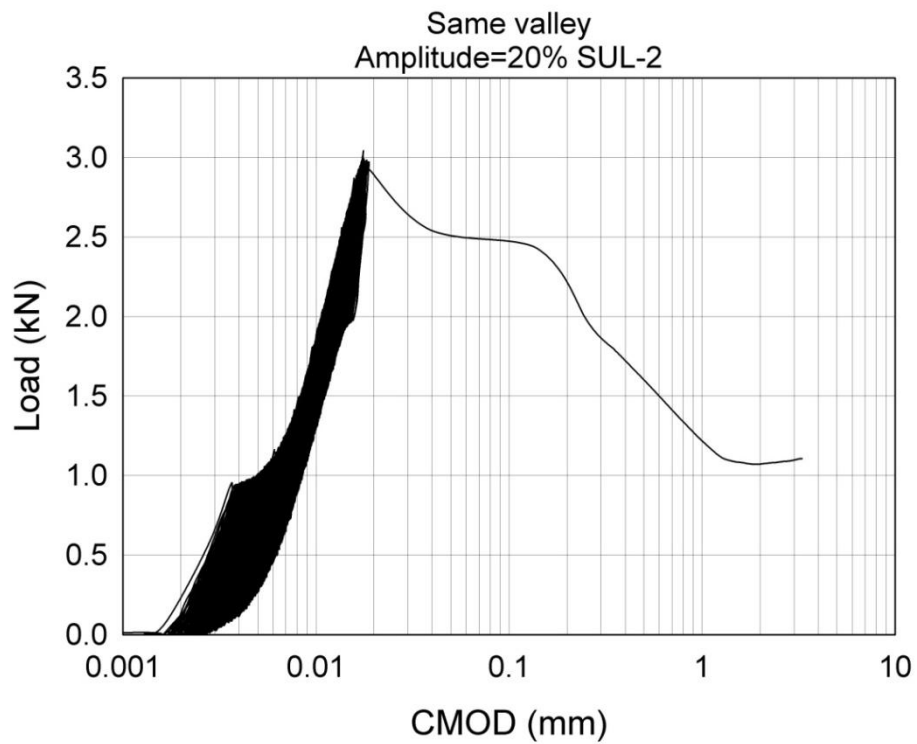


(b)

Figure 7.7 Load-CMOD plots of Type I cyclic loading with amplitude = 10% SUL (0.45 kN)

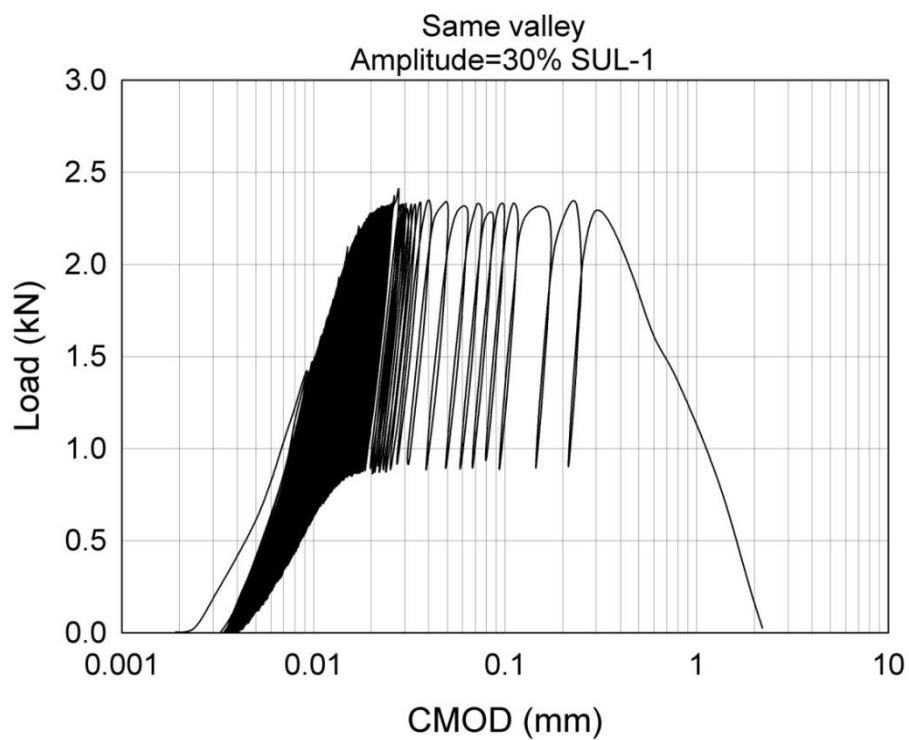


(a)

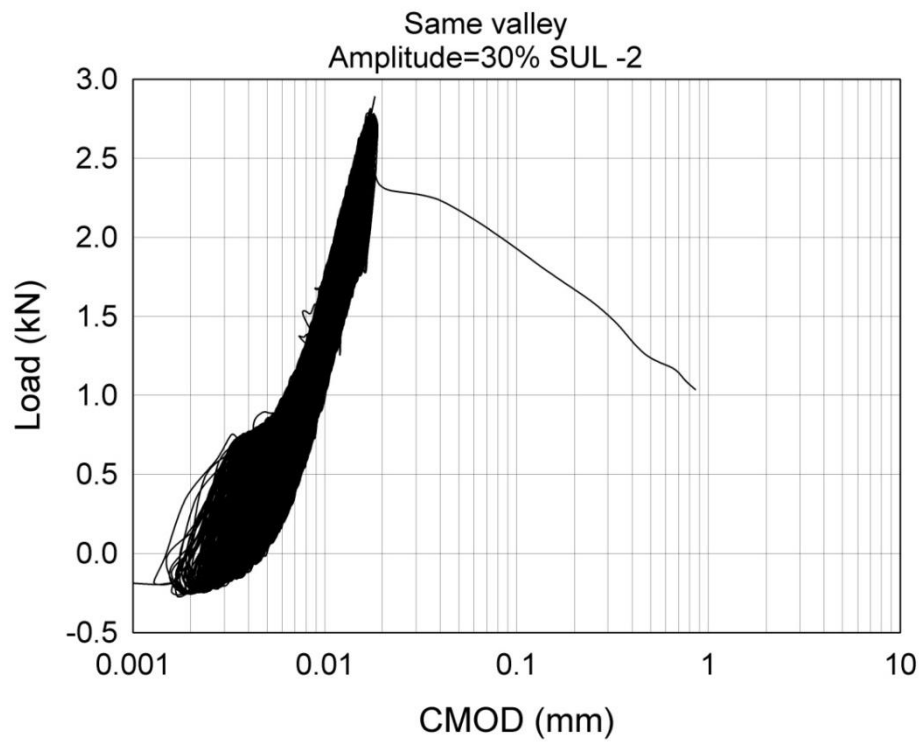


(b)

Figure 7.8 Load–CMOD plots of Type I cyclic loading with amplitude = 20% SUL (0.9 kN)

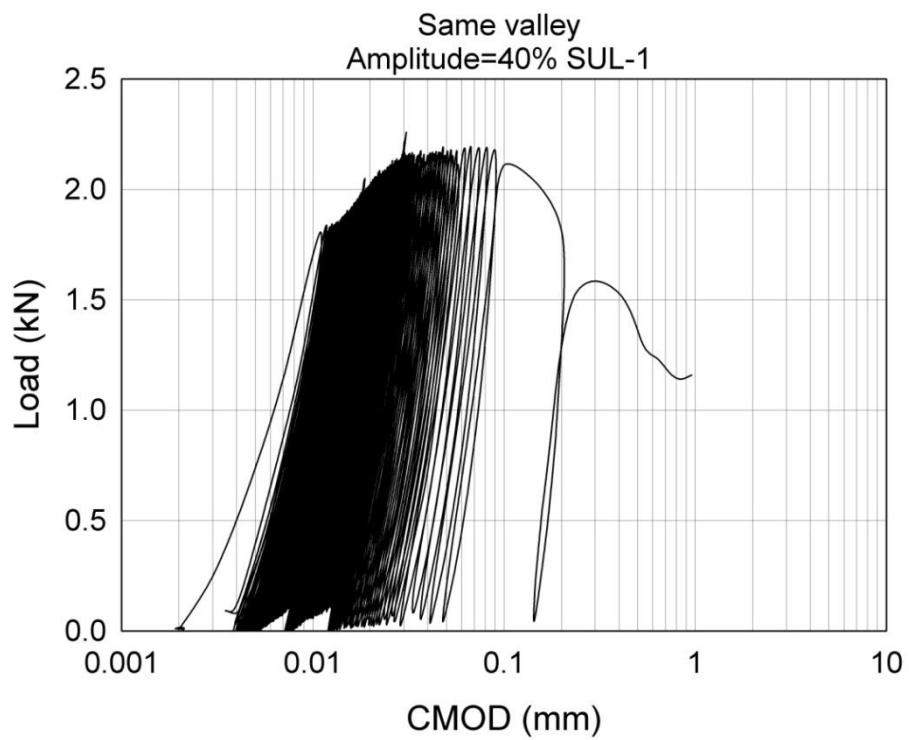


(a)

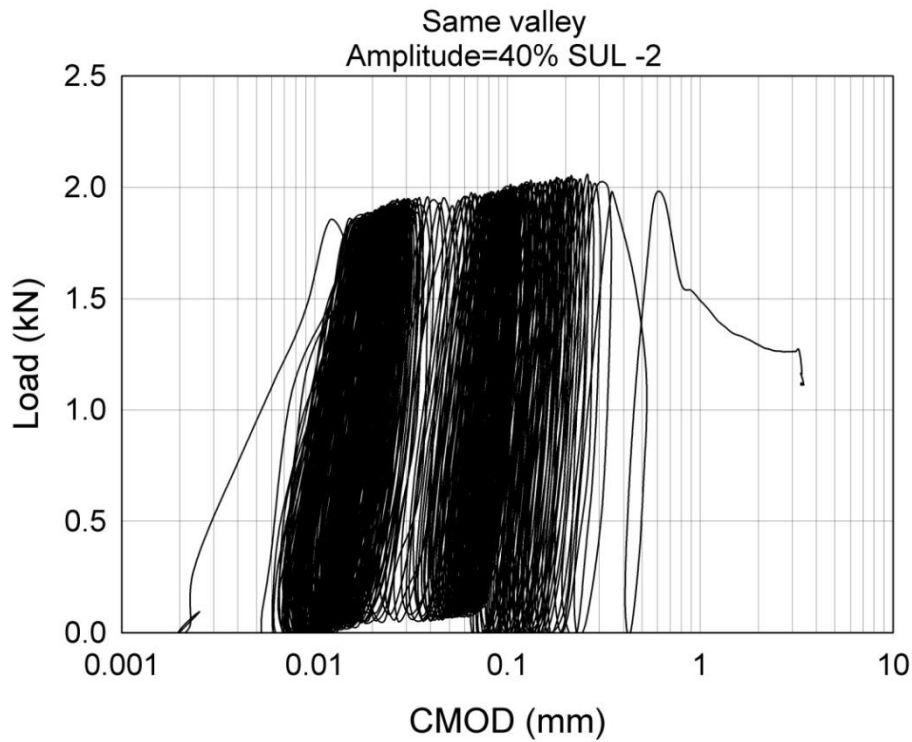


(b)

Figure 7.9 Load-CMOD plots of Type I cyclic loading with amplitude = 30% SUL (1.35 kN)



(a)



(b)

Figure 7.10 Load–CMOD plots of Type I cyclic loading with amplitude = 40% SUL (1.8 kN)

7.2.1.1 Damage evaluation curves

A typical feature of rock fatigue in experimental tests can be observed by producing a progressive accumulation of permanent strain in the specimen, rather than any significant decay in the material's elastic modulus. Accumulation of plastic deformation is responsible for the fatigue damage; the magnitude and increasing trend of the irreversible deformation influence the cumulative fatigue damage. The displacements along both x and y directions represent CMOD and diametral axial displacement, respectively, as shown in Figures 7.11, 7.12, 7.13 and 7.14 with different amplitudes. As shown in the damage plots, cyclic loading had a significant effect on rock strength degradation with continuous accumulation of irreversible deformation up to failure. From displacement responses in both directions, it can be seen that irreversible deformation increases continuously up to failure. At the beginning, irreversible deformation develops fast, then deformation increases at a slow constant rate and finally cumulative deformation begins to grow very fast until failure. In recent research, three typical damage accumulation phases were observed in studies of cylindrical samples under uniaxial compressive cyclic loading (Xiao *et al.*, 2009; Zang *et al.*, 2008). As shown in Figures 7.11, 7.12, 7.13 and 7.14, where we did not plot the maximum diametral axial displacement at the top of each cycle ($\epsilon_{\max}$) and minimum at the bottom of each

cycle ($\varepsilon_{\min}$) separately, the behaviour of both $\varepsilon_{\max}$ and $\varepsilon_{\min}$ is similar to the cumulative dilatant creep behaviour of rocks under static fatigue loading conditions. $\varepsilon_{\max}$ represents the cumulative development of dilatancy during cyclic loading, whereas $\varepsilon_{\min}$ represents the accumulation of ‘permanent’ dilatancy at the bottom of each cycle after each unloading. In contrast, the difference between $\varepsilon_{\max}$ and $\varepsilon_{\min}$ is almost constant up to failure, whereas the difference between CMOD readings at the top and bottom of each cycle are decreasing up to failure. In general, the minimum CMOD reading at the bottom of each cycle behaves more similar to the dilatant creep behaviour than does the maximum CMOD reading at the top of each cycle.

It is interesting to note that the CMOD is very small, $0.02 \text{ mm} < \text{CMOD} < 0.04 \text{ mm}$, compared with that of diametral axial displacement, $0.1 \text{ mm} < \text{diametral axial displacement} < 0.16 \text{ mm}$, before unstable crack propagation starts. This shows that there is stable crack propagation (subcritical crack propagation) in front of the tip of the chevron notch crack, thus, the CMOD reading of the outer crack mouth is not affected much by this stable crack propagation.

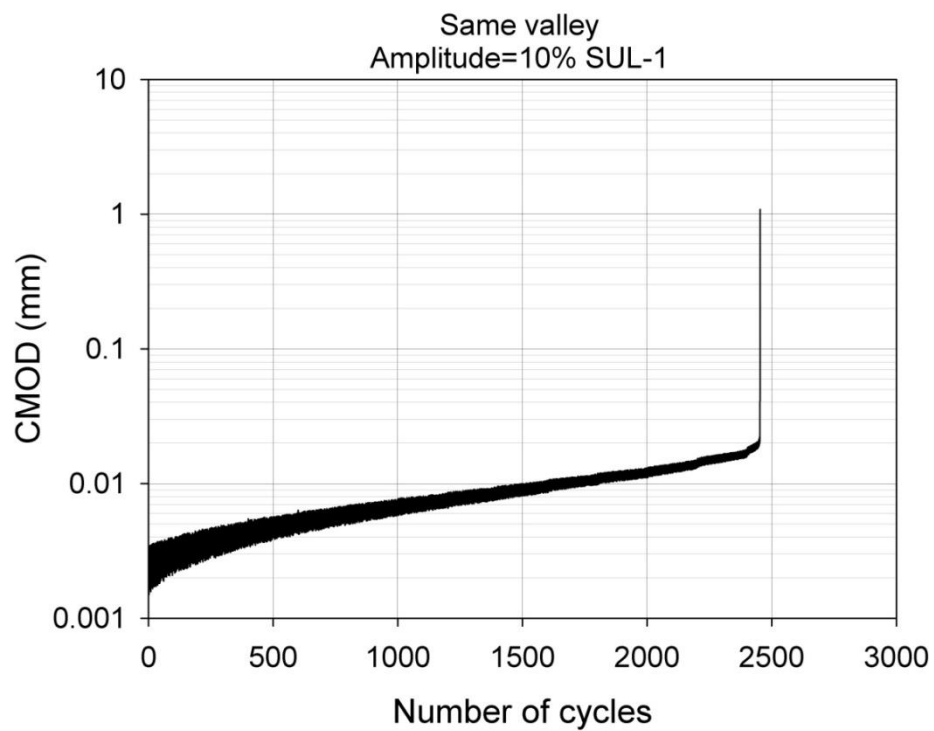
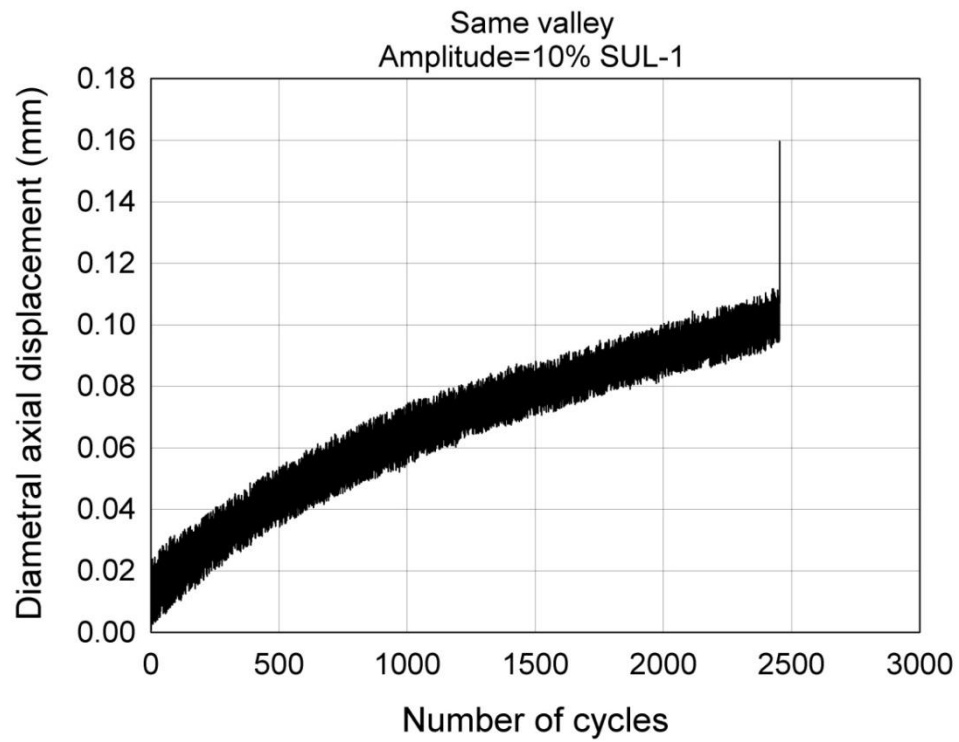


Figure 7.11 Diametral axial displacement and CMOD versus number of cycles plots; Type I cyclic loading tests with amplitude 10% SUL (0.45 kN)

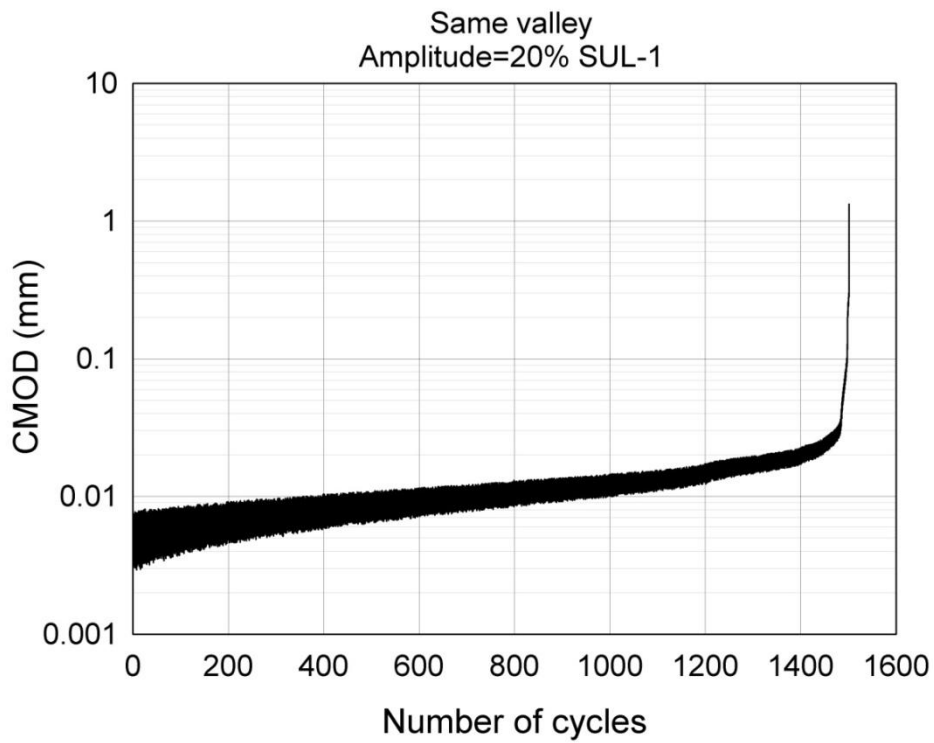
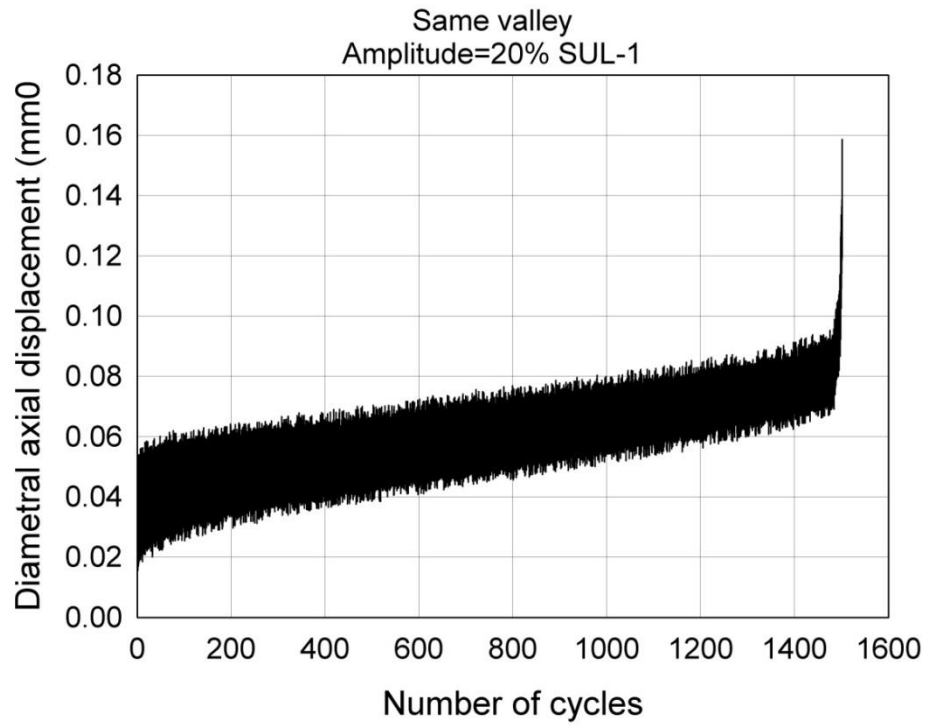


Figure 7.12 Diametral axial displacement and CMOD versus number of cycles plots; Type I cyclic loading tests with amplitude 20% SUL (0.9 kN)

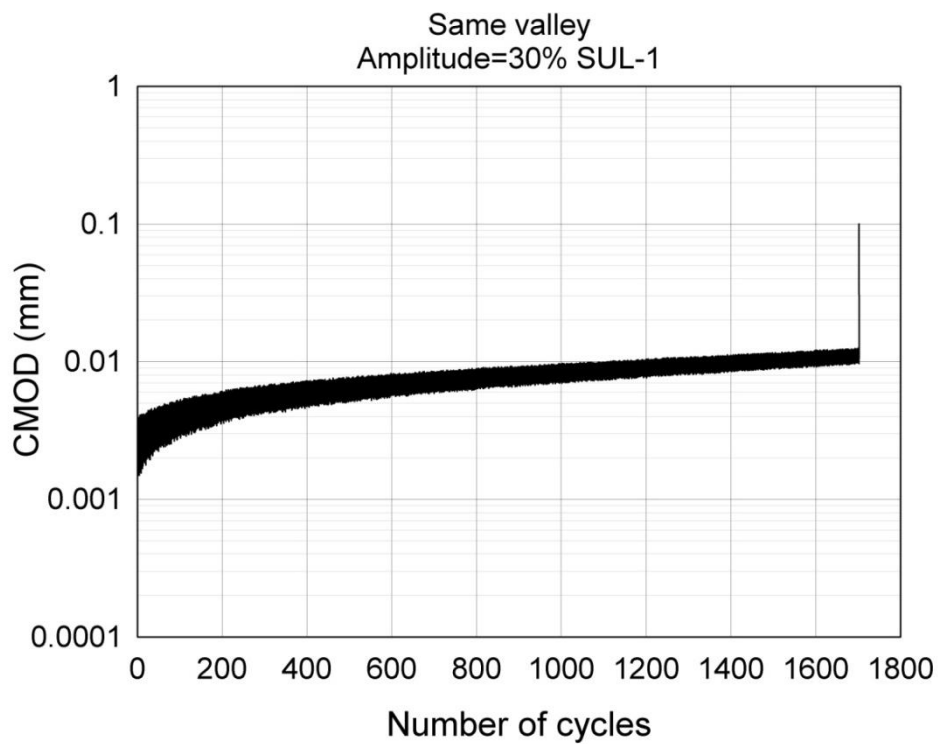
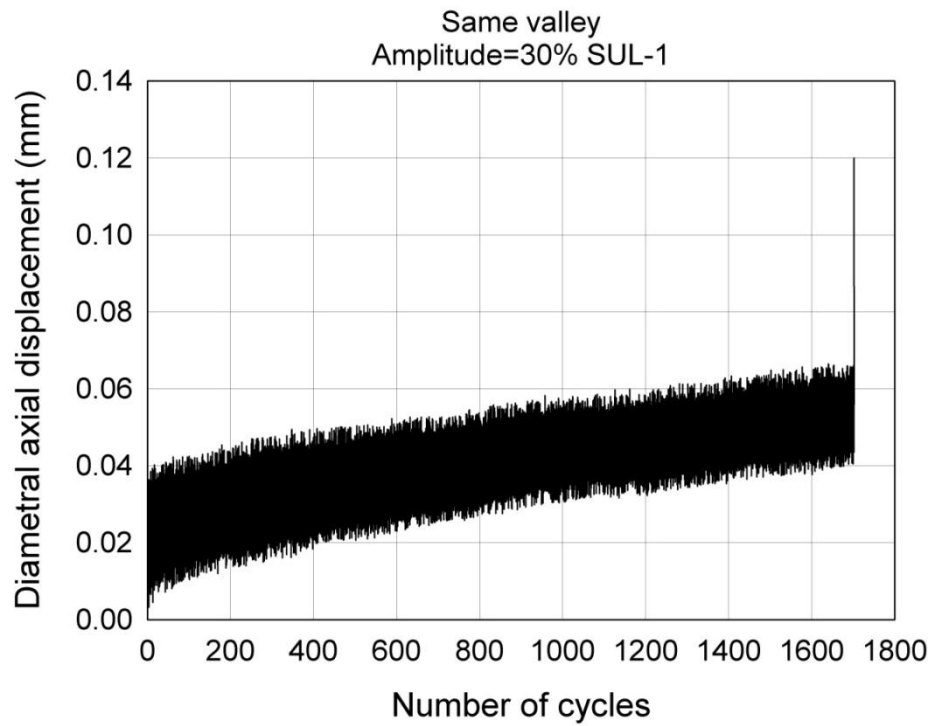


Figure 7.13 Diametral axial displacement and CMOD versus number of cycles plots; Type I cyclic loading tests with amplitude 30% SUL (1.35 kN)

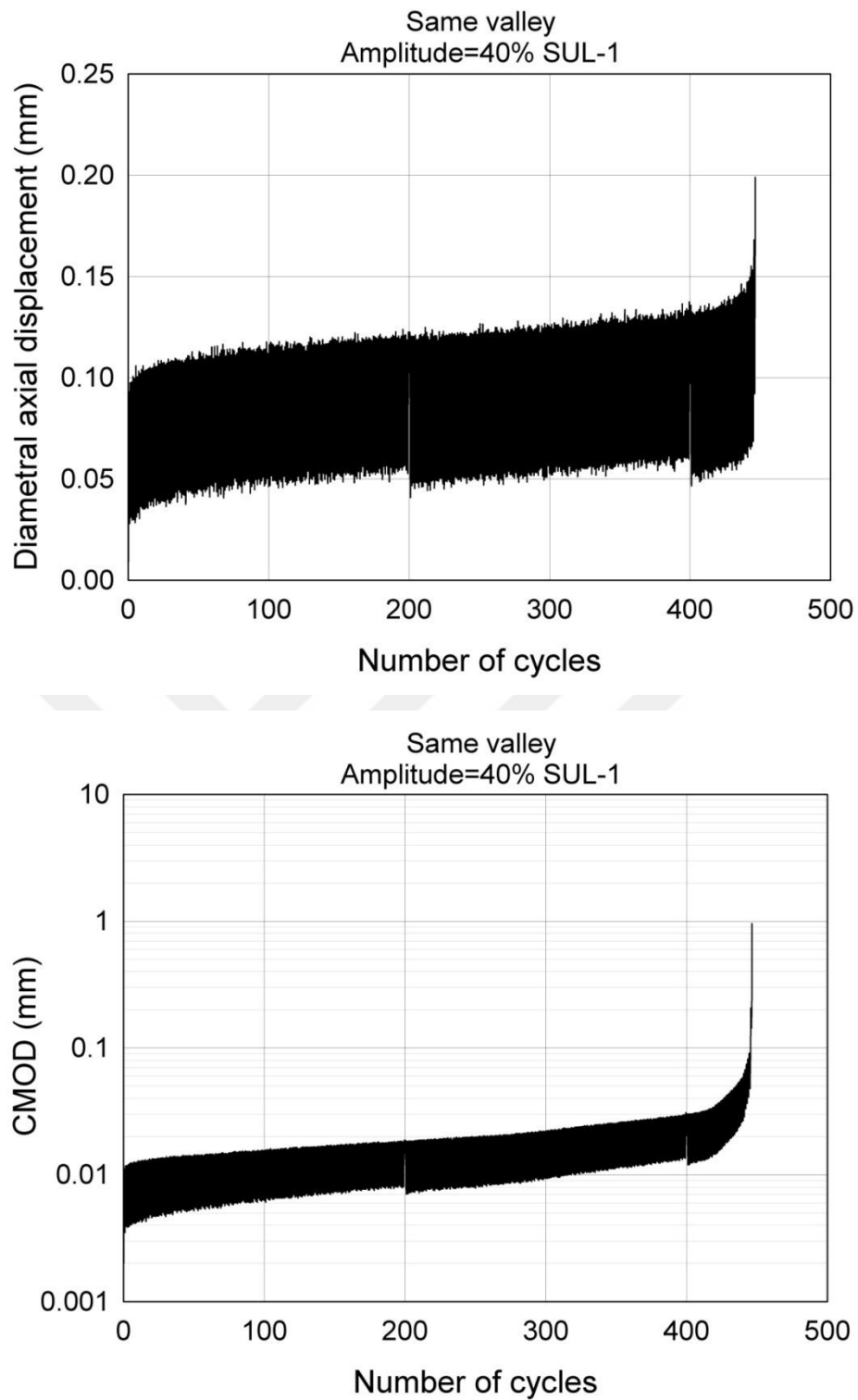


Figure 7.14 Diametral axial displacement and CMOD versus number of cycles plots; Type I cyclic loading tests with amplitude 40% SUL (1.8 kN)

7.2.2 Effect of Cyclic Loading with Increasing Mean Values on Mode I Fracture Toughness

The constant increasing rate mean value cyclic loading test, termed Type II cyclic loading test, was performed whereby a CCNBD specimen of Brisbane tuff was loaded in diametral cyclic

compression to a load level just above the level of the previous cycle with constant amplitude and constant increasing mean value up to failure. The difference in this type of cyclic load from the Type I cyclic loading test is the four different amplitude test series have the same mean values with increasing rate instead of increasing valley values. Peak load values for Type II cycles are higher than peak load values for Type I cycles in test series with small amplitudes. The main purpose of using two different incremental cyclic tests is to investigate the mechanics of deformation under the consideration of the load level of each unloading cycle. Because the valley value at which each unloading cycle stops affects the crack surface energy balance in the material, the remaining energy input that satisfies the conditions for damage accumulation are expected to be different.

Fracture toughness values of Brisbane tuff under static loading were determined in the previous section as $1.12\text{--}1.5 \text{ MPa}\sqrt{\text{m}}$ using ISRM (1978) suggested methods. Results from Type II cyclic loading tests are given in Table 7.3. In general, the failure load and number of cycles decrease with increasing amplitude. According to the obtained results, the maximum reduction, 42% of static K_{IC} , was obtained with the highest amplitude, 1.8 kN (40% SUL) under cyclic loading. Mode I fracture toughness values of Brisbane tuff (NST50) are reduced from $1.12 \text{ MPa}\sqrt{\text{m}}$ to $0.65 \text{ MPa}\sqrt{\text{m}}$ under highest amplitude increasing cyclic loading. This reduction of fracture toughness with the second type of increasing cyclic loading is slightly lower than the obtained reduction with the first type of increasing cyclic loading.

Table 7.3 Results of Type II cyclic loading tests on CCNBD samples

Sample	Amplitude % SUL	Ultimate load (kN)	Number of cycles up to failure	Mode-I Stress Intensity Factor (K_I) [*] $MPa\sqrt{m}$ (Shetty, 1985)	Fracture toughness (K_{IC}) ^{**} $MPa\sqrt{m}$	Reduction of K_{IC} (%)
CCNBD-Rp1	10	2.67	2261	0.67	0.83	26
CCNBD-Rp2	10	2.59	2084	0.65	0.80	29
CCNBD-Rp3	10	2.06	1437	0.53	0.64	43
CCNBD-Rp4	10	3.19	2449	0.82	0.98	13
CCNBD-Rp5	10	3.15	2404	0.81	0.97	14
Average		2.82	2208	0.70	0.84	25
CCNBD-Rp1	20	3.12	2493	0.80	0.96	15
CCNBD-Rp2	20	2.64	1885	0.68	0.81	28
CCNBD-Rp3	20	2.59	1823	0.66	0.80	29
CCNBD-Rp4	20	1.97	987	0.51	0.61	46
Average		2.68	1949	0.66	0.79	30
CCNBD-Rp1	30	2.40	1237	0.61	0.74	34
CCNBD-Rp2	30	2.07	869	0.53	0.64	43
CCNBD-Rp3	30	2.70	1645	0.69	0.83	26
CCNBD-Rp4	30	2.41	1240	0.62	0.74	34
Average		2.39	1247	0.61	0.73	34
CCNBD-Rp1	40	2.32	399	0.59	0.71	37
CCNBD-Rp2	40	1.95	176	0.50	0.61	46
CCNBD-Rp3	40	2.04	243	0.52	0.63	44
CCNBD-Rp4	40	2.17	446	0.56	0.67	41
Average		2.12	306	0.54	0.65	42

* α is assumed as 0.5, ** Y_{min}^* is assumed as 1.21

A comparison between static and dynamic cyclic loading tests is shown in Figure 7.15 by plotting both results on the same axes. Amplitude of the dynamic cyclic loading used in the test was 0.45 kN, 10% of SUL. As seen from Figure 7.15, both failure load values and damage mechanisms are quite different under static and dynamic cyclic loading. The failure load obtained from the static test was reduced from 4.3 kN to 2.59 kN because of rock fatigue. In both test types, stable and unstable

crack propagation stages are clear, however, the resistance of crack propagation to cyclic loading with accumulation of plastic deformation (0.3 mm) is much greater compared with the relative value (0.023 mm) under static loading before failure (Figure 7.15). This behaviour shows that development of a large number of microcracks causing accumulation of irreversible deformation occurred in front of the tip of the chevron notch crack. Traditionally, damage has been quantified as the accumulation of permanent strain with each cycle, because microfracturing introduces an element of nonlinearity into the theoretical elastic behaviour of the sample. It was therefore seen that damage did not take place in samples before failure under static loading, however, a large scale damaged zone occurred in front of the chevron notch crack tip under cyclic loading.

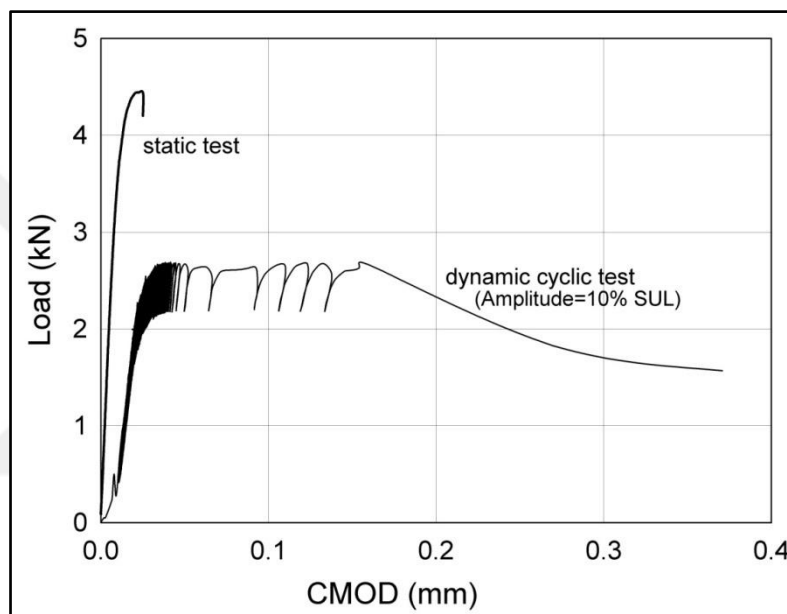


Figure 7.15 Comparison of load–CMOD curves of CCNBD specimens tested under static and Type II cyclic loading

There is a clear crushed region including small particles and dust in front of the chevron tip in CCNBD samples failed under cyclic loading, as shown in Figure 7.16b. However, any rock chips (small particles) were not observed at crack surfaces of failed specimens under static loading (Figure 7.16a). It was seen from the experimental outputs that there is effective accumulated plastic deformation prior to failure under cyclic loading, as shown in Figure 7.15. This behaviour shows that the crack tip has become blunted as a result of the formation of microcracking. This damaged zone in front of the chevron tip is called the FPZ. According to Nonlinear Elastic Fracture Mechanics (NEFM), a significant FPZ is expected in front of the tip of the notch crack because load–CMOD plots show extensive plastic deformation before failure (Whittaker *et al.*, 1996).

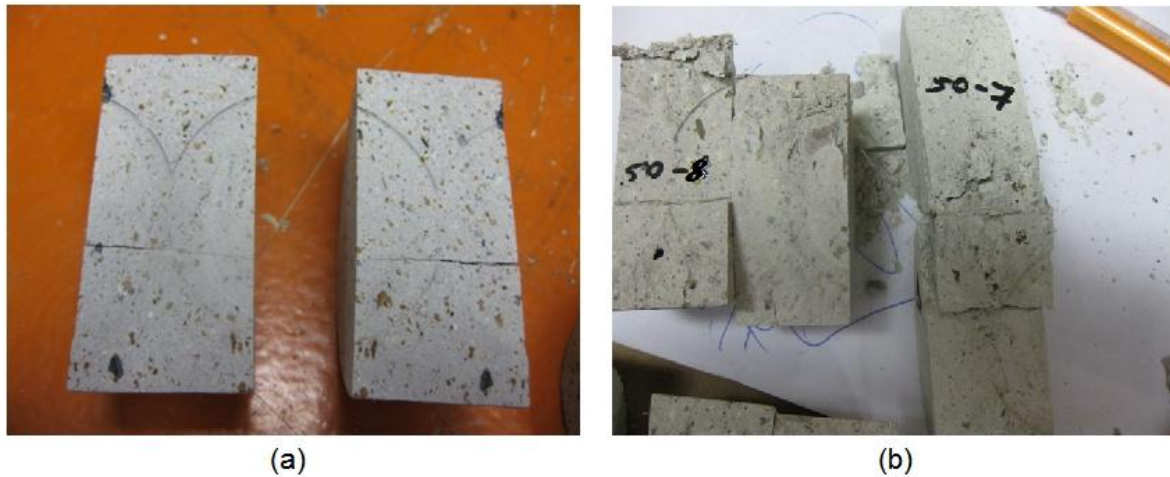


Figure 7.16 Failed specimens and damaged zone in front of the chevron tip (a) under static loading and (b) under Type II cyclic loading

Figures 7.17–7.20 show the load–CMOD plots at different amplitudes of Type II cyclic loading tests. The loading–unloading frequency was 1 Hz. Because the plastic deformation is very high compared with elastic deformation, the CMOD axis was plotted as a log scale. A clear tensile softening can be seen in all plots through postpeak behaviour in load–CMOD plots after an accumulated plastic deformation in the samples. According to classical fracture mechanics, stable crack extension with plastic strain accumulation is possible under indirect tensile cyclic loading even with various amplitude values, as shown in all plots. However, it is hard to say there is a single crack plane, as defined traditionally under static loading in fracture mechanics, which takes place in front of the chevron tip under cyclic loading. Hence, ‘damage’ is the preferred term to use in rock fatigue research instead ‘failure plane’ (main crack). It is therefore possible to see quantified damage as the accumulation of permanent strain with each cycle in all load–CMOD plots.

Similar to the Type I cyclic tests, any elastic deformation did not take place before the start of plastic deformation in the highest amplitude cyclic loading tests (Figure 7.20). The elastic deformation region was seen to gradually decrease with increasing amplitude of dynamic cyclic loading.

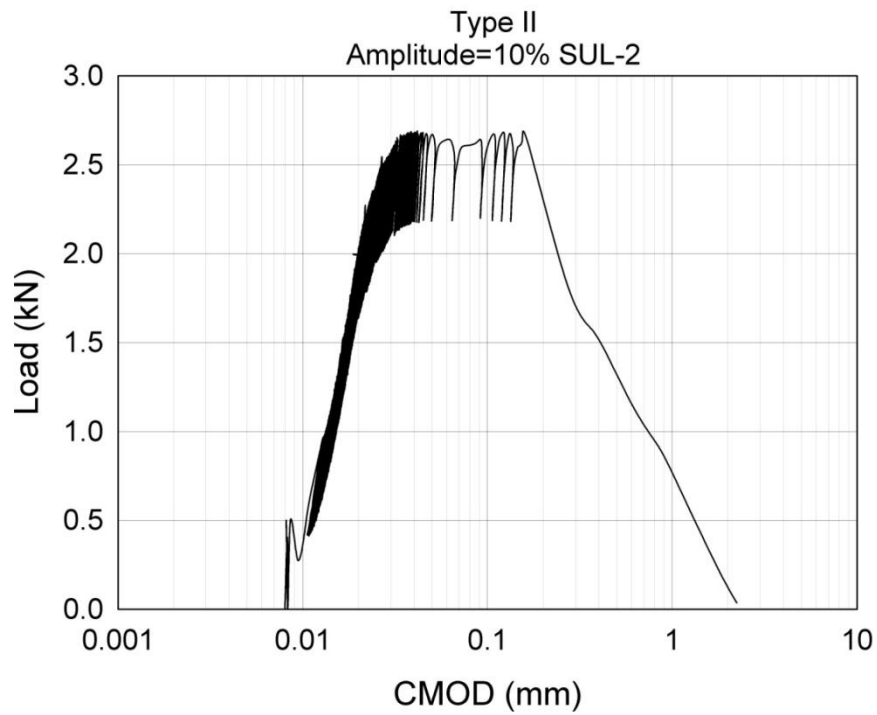
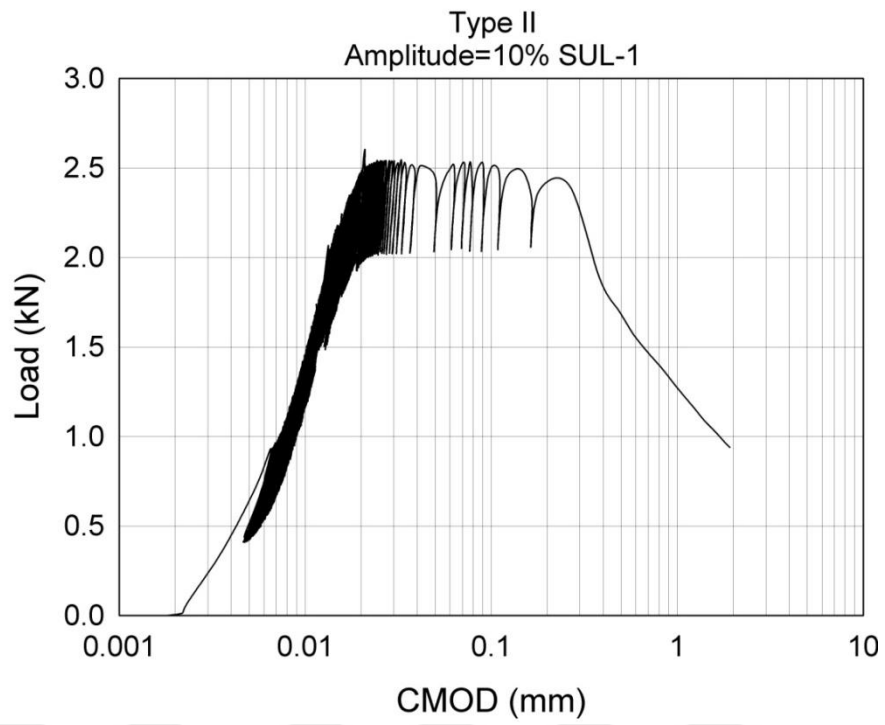


Figure 7.17 Load–CMOD plots of Type II cyclic loading with amplitude = 10% SUL (0.45 kN)

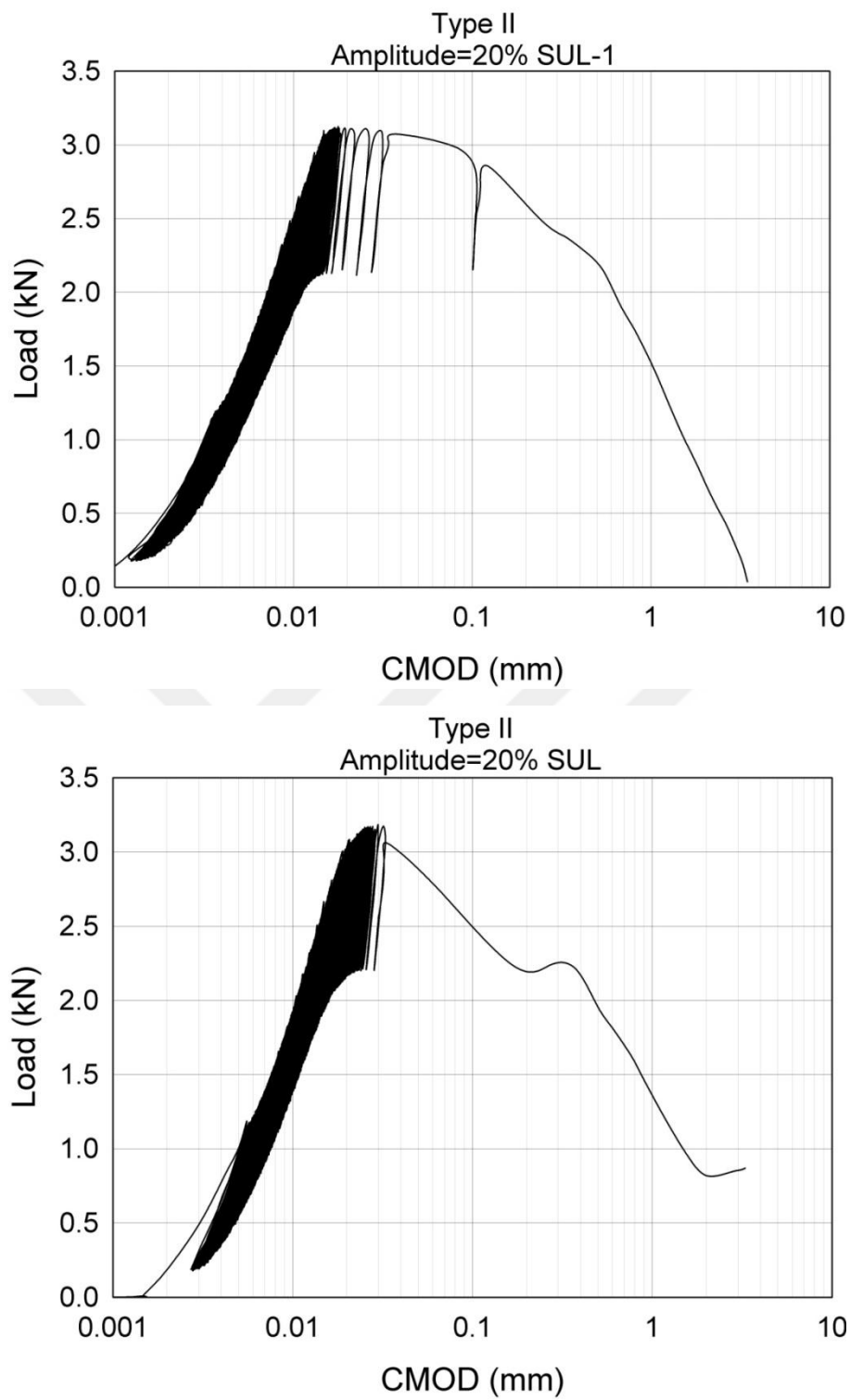


Figure 7.18 Load-CMOD plots of Type II cyclic loading with amplitude = 20% SUL (0.9 kN)

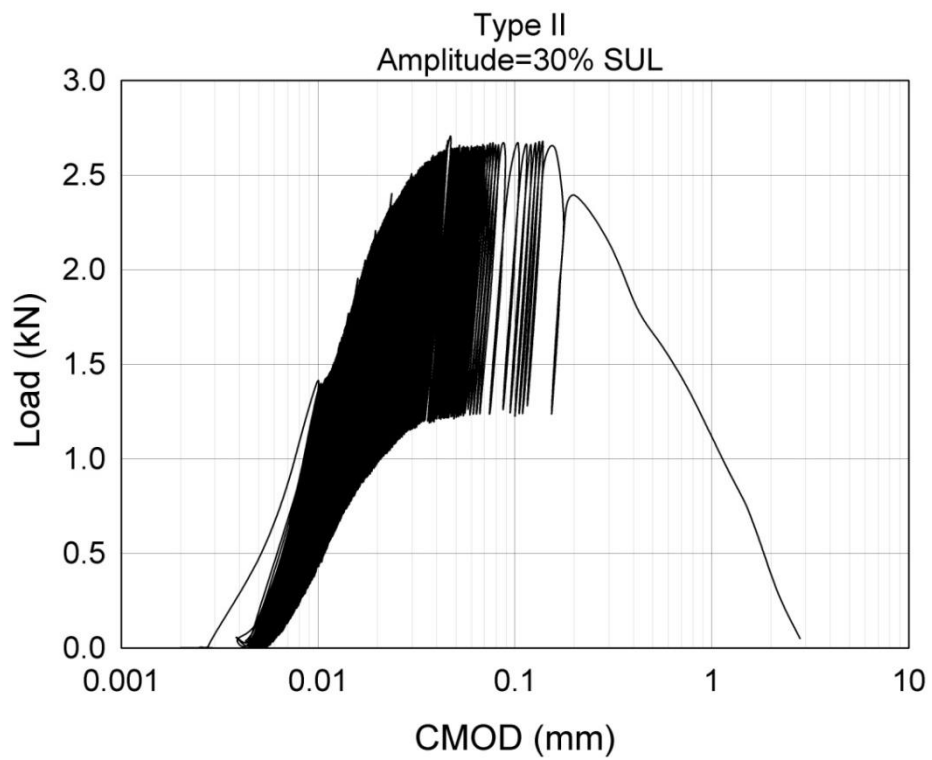
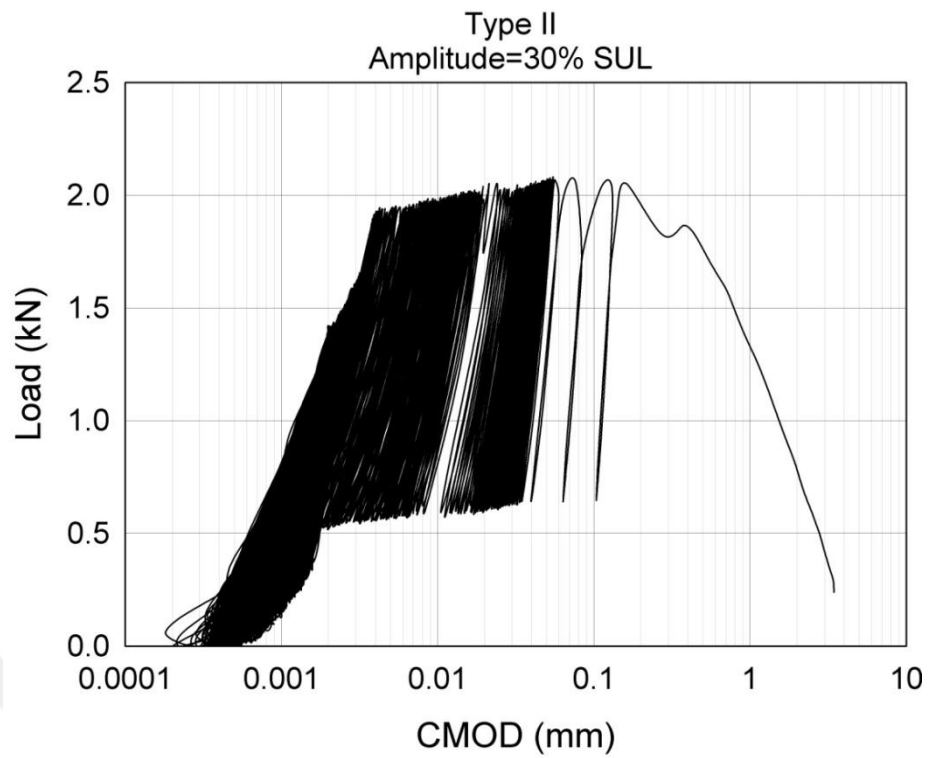


Figure 7.19 Load–CMOD plots of Type II cyclic loading with amplitude = 30% SUL (1.35 kN)

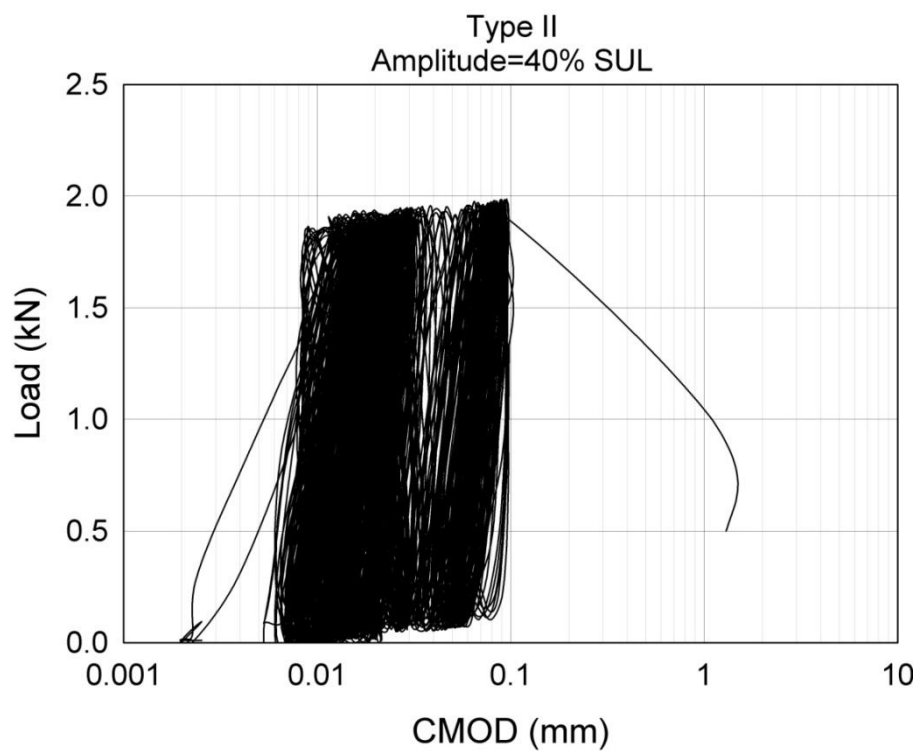
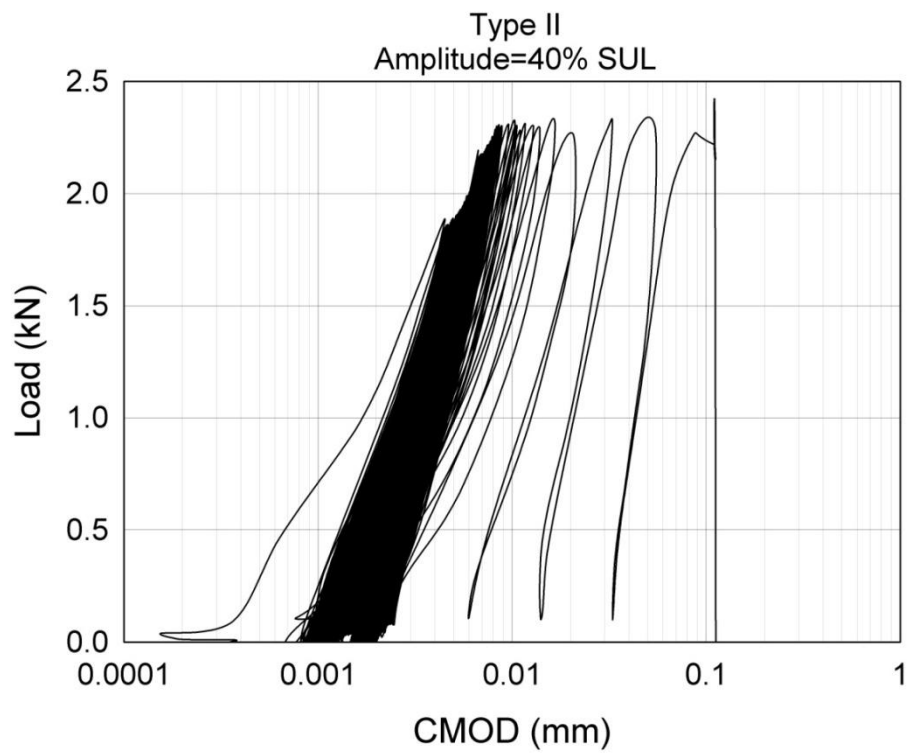


Figure 7.20 Load–CMOD plots of Type II cyclic loading with amplitude = 40% SUL (1.8 kN)

7.2.2.1 Damage evaluation curves

Results from this test indicate that with each cycle and load increment additional damage is incurred in the sample (Figure 7.21–7.24). Plots of permanent damage show that both the CMOD

and diametral axial displacement increases with increasing damage increments but at different rates. Accumulation of plastic deformation is responsible for the fatigue damage; the magnitude and increasing trend of the irreversible deformation influence the cumulative fatigue damage. As shown in the damage plots, cyclic loading did have a significant effect on rock strength degradation with continuous accumulation of irreversible deformation up to failure.

The behaviour of CMOD and diametral axial displacement up to failure demonstrates that the irreversible deformation increases continuously up to failure. However, damage accumulation in both directions, parallel to the loading axis and normal to the loading axis, is different. At the beginning, irreversible CMOD deformation develops fast, then deformation increases at a slow constant rate and finally cumulative deformation begins to grow very fast until failure. In contrast, diametral axial displacement response to Type II cyclic loading is different from the response to Type I cyclic loading. All four different amplitude test results show that diametral axial displacement response increases with a concave downward behaviour. Moreover, the difference in diametral axial displacement between peak and valley values of each cycle is almost same up to failure while the difference in CMOD displacement between peak and valley values of each cycle is decreasing up to failure. The opening of crack faces parallel to the applied load and the closure of crack faces perpendicular to the load cause certain changes in the relative lateral and axial deformations, respectively. These changes appear as inflections in the stress–strain curves that, in turn, can be used to identify the different stages of rock deformation and failure. Diametral axial displacement–number of cycles plots show that the displacement rate increases with an increasing rate up to failure (Figures 7.21–7.24). This result may show that irreversible damage accumulation in front of the chevron tip is not caused by the closing of pre-existing microcracks aligned parallel to the maximum principal stress.

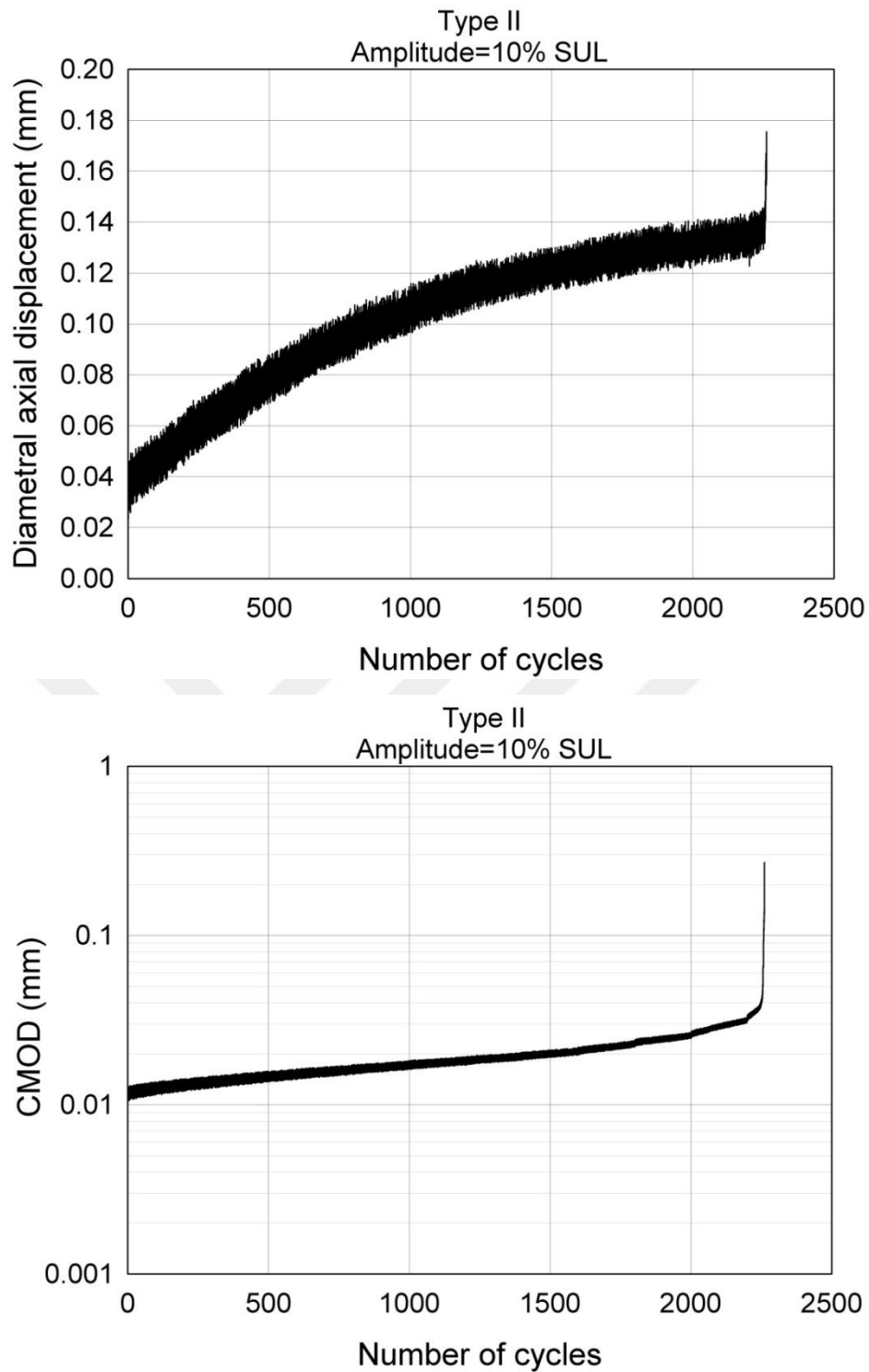


Figure 7.21 Diametral axial displacement and CMOD versus number of cycles plots; Type II cyclic loading tests with amplitude 10% SUL (0.45 kN)

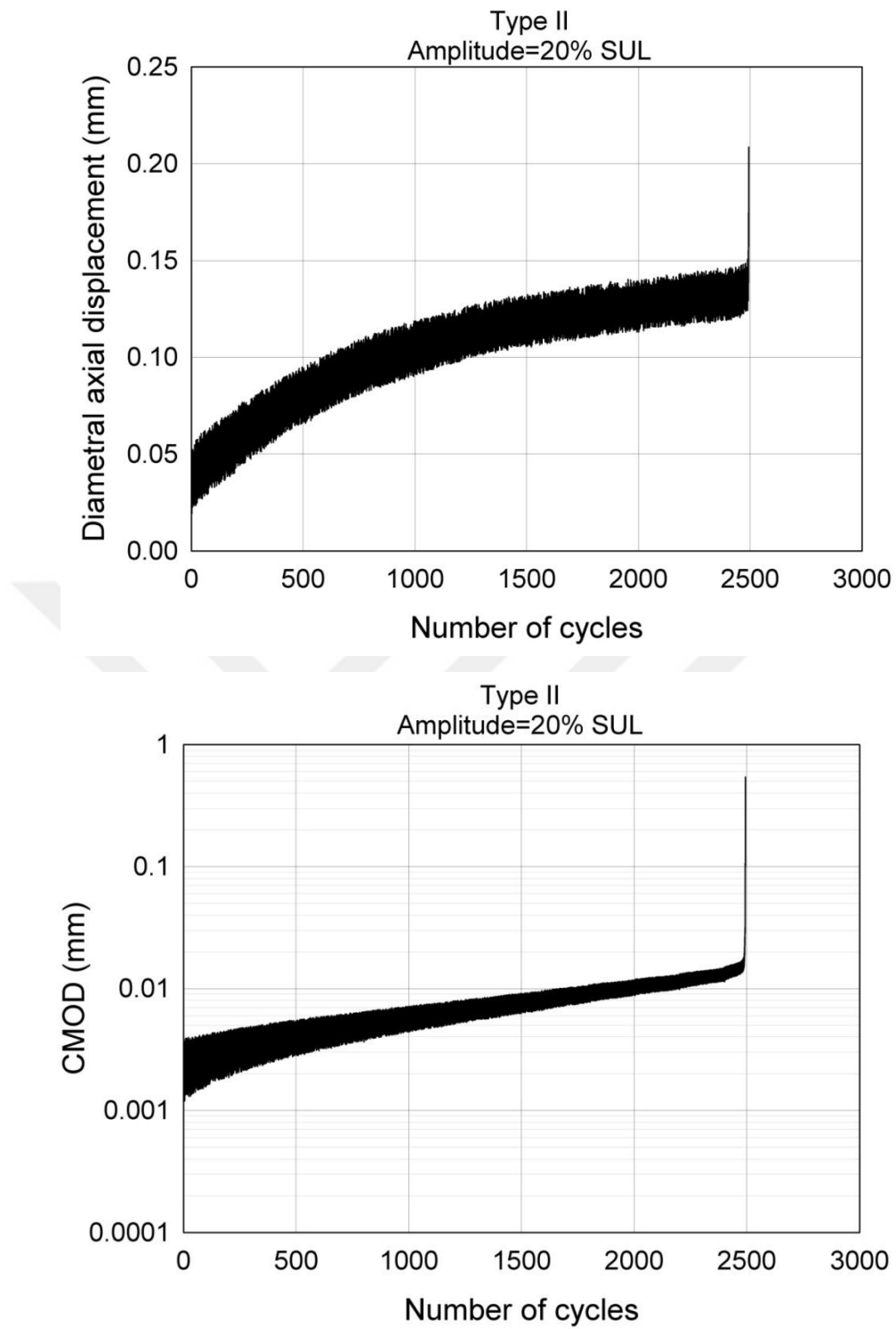


Figure 7.22 Diametral axial displacement and CMOD versus number of cycles plots; Type II cyclic loading tests with amplitude 20% SUL (0.9 kN)

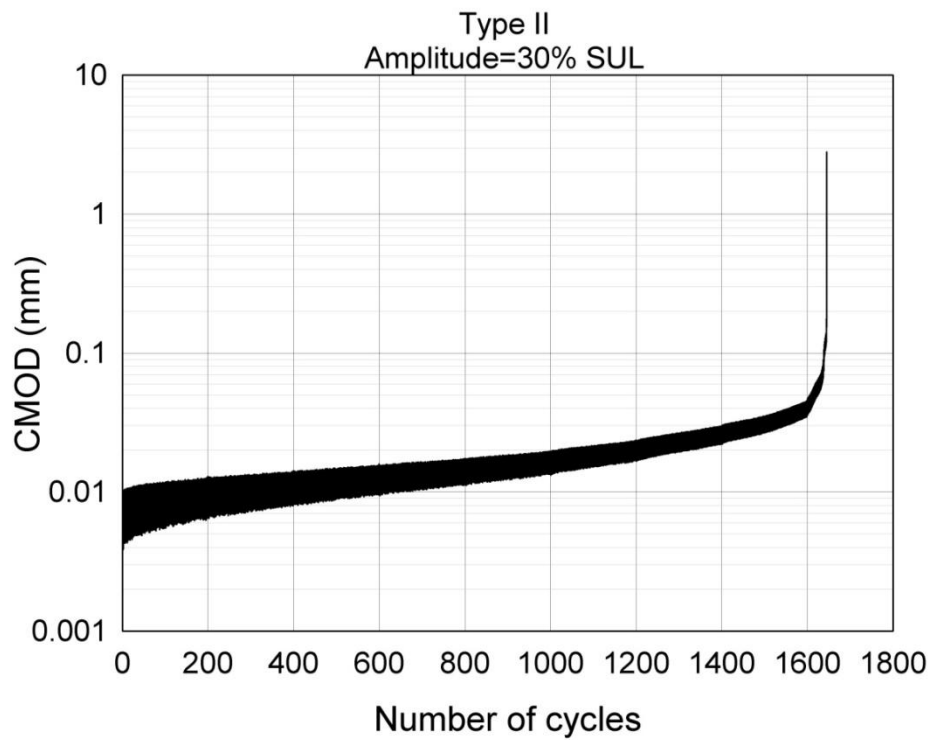
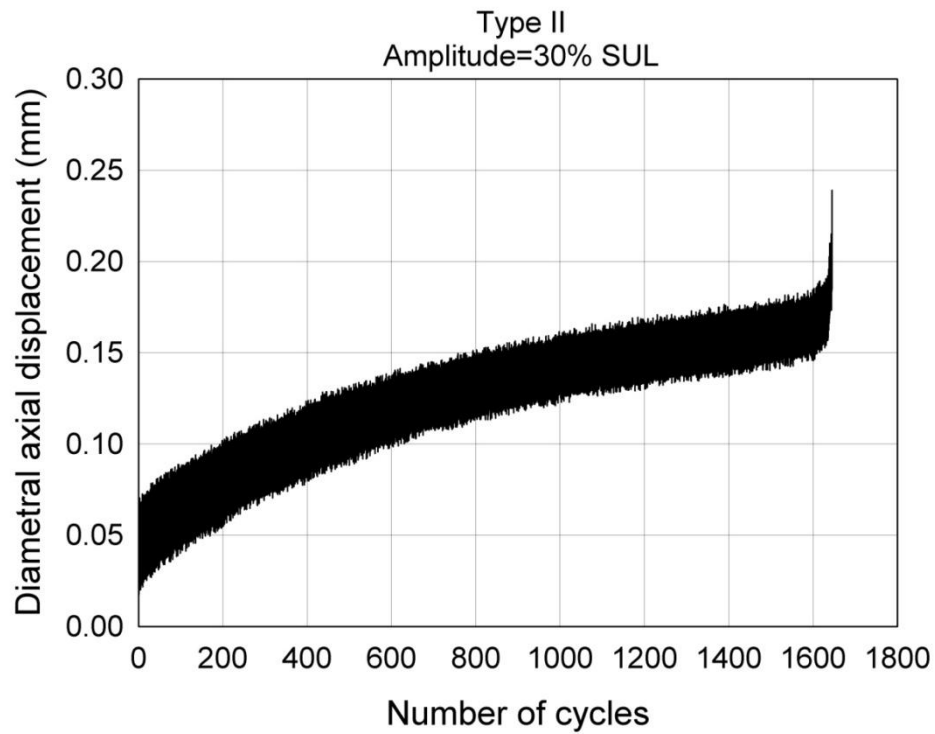


Figure 7.23 Diametral axial displacement and CMOD versus number of cycles plots; Type II cyclic loading tests with amplitude 30% SUL (1.35 kN)

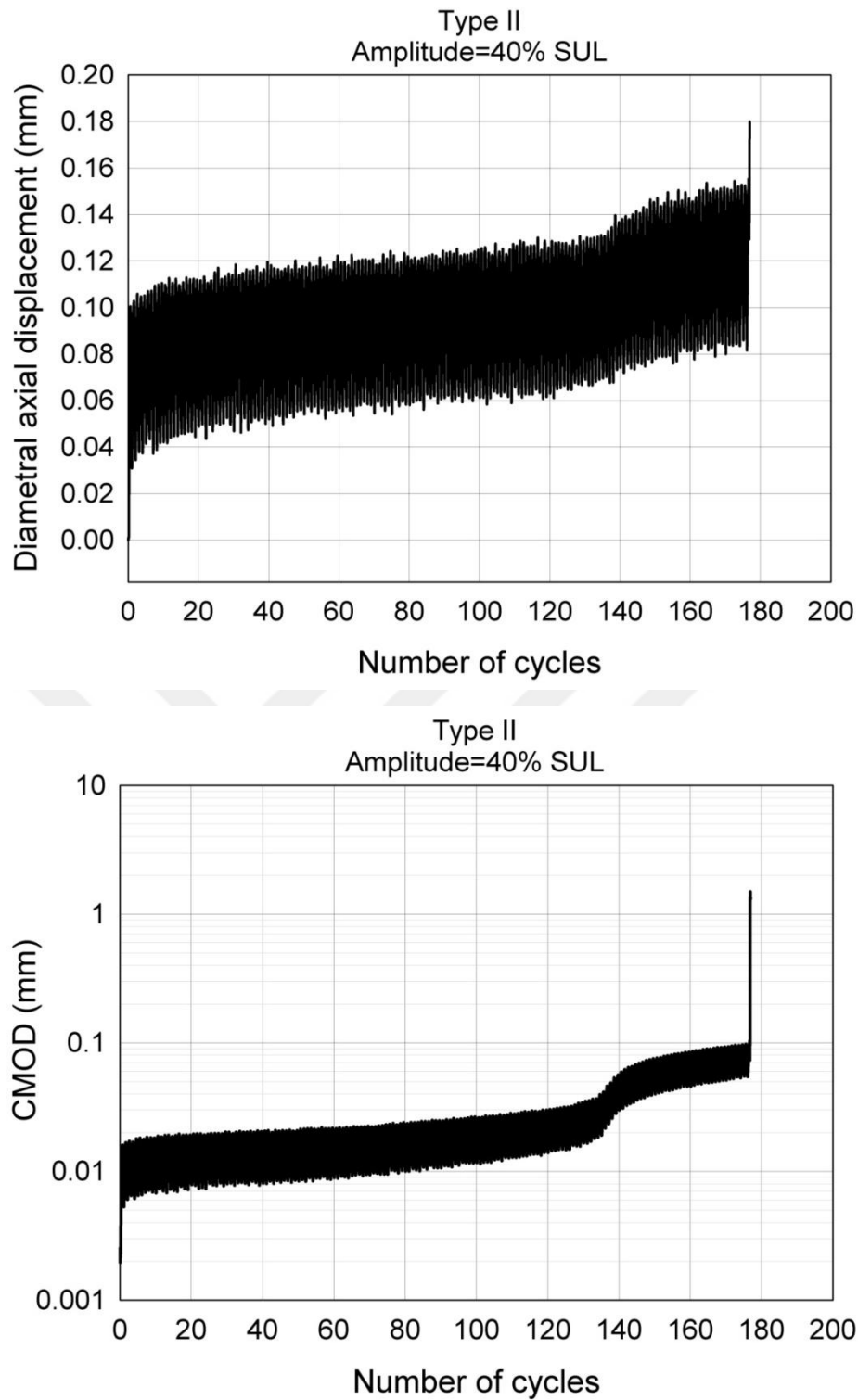


Figure 7.24 Diametral axial displacement and CMOD versus number of cycles plots; Type II cyclic loading tests with amplitude 40% SUL (1.35 kN)

A closer examination of diametral axial displacement and CMOD after the start of nonlinear behaviour in plots shows both diametral axial displacement and CMOD increase with a concave upward rate (Figure 7.25). It is definitely clear that the rate of cracking gets faster after this point. Continuous irreversible damage happens slowly up to this point because new stress-induced

microcracks take place up to this point. It is possible to say that coalescence of the induced cracks starts after this point, up to failure.

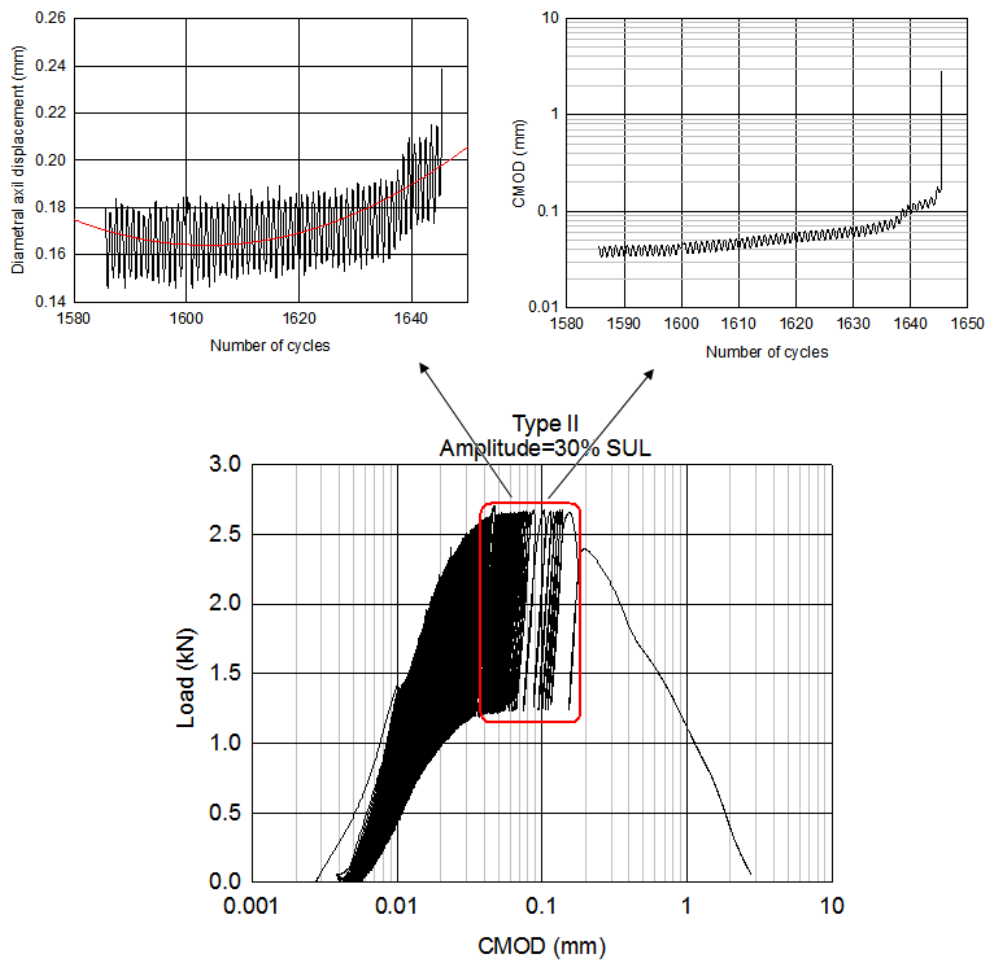


Figure 7.25 Diametral axial displacement and CMOD after the start of nonlinear deformation in front of the chevron crack tip

7.3 Behaviour of Inclined Chevron Notched Crack under Cyclic Loading

Mixed-mode I–II fracture problems in compression are shown to be more complicated and also quite different from those under tension. It is accepted in general that tensile cracks initially grow at an angle with respect to the direction of axial compressive stress, then rapidly grow along the axial compressive stress (Al-Shayea, 2005; Hoek & Bienawskie, 1965; Li *et al.*, 2005). Relatively few published results are available for experimental work on mixed-mode I–II fracturing in rocks under static loadings. In contrast, there has not been any experimental or numerical research on mixed-mode rock fracturing under cyclic loading in literature.

More commonly, if a CCNBD specimen is loaded with the crack orienting at an appropriate angle (β) with respect to the loading direction, a mixed-mode I–II stress field can be achieved at the crack tips to initiate and propagate wing cracks inclined at an angle, θ , to the crack plane (Figure 7.26). Hence, CCNBD specimens were used to investigate mixed-mode rock fracturing under cyclic loading in this research.

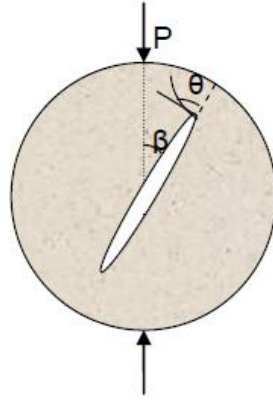


Figure 7.26 CCNBD sample with an inclined crack under diametral compression

The mixed-mode loading condition for a crack plane in CCNBD specimens will be compression-shear like loading. A negative mode I stress intensity factor, K_I , means compressive stress acting at the crack tip, and a negative mode II stress intensity factor, K_{II} , just indicates an opposite direction of the shear stress acting parallel to the crack plane. In this case, a positive crack inclination angle (θ) for compression means that the crack propagation direction tends to be parallel to the direction of loading or normal to the direction of the minor principal stress. In contrast, the crack path extends perpendicular to the direction of loading and, hence, a negative θ is obtained while in tension. Further information about mixed-mode stress intensity factors and fracturing was given in previous chapters.

A series of diametral compressive dynamic cyclic loading tests was performed on 27 CCNBD Brisbane tuff samples. The chevron notch cracks inclined at three different angles— $\beta = 30^\circ$, $\beta = 45^\circ$ and $\beta = 60^\circ$ —were tested under three different amplitudes: 0.45 kN, 0.9 kN and 1.35 kN. Load and CMOD values were recorded continuously up to failure of the specimens. Failure load and relative number of cycles for each inclined crack tested under various amplitudes of cyclic loading are shown in Table 7.4. In general, the results show that the failure load obtained with cyclic loading tests decreased between 30% and 45% compared with relative values obtained under static loading tests. Values in Table 7.4 show that only a minor failure load increase was obtained with $\beta = 45^\circ$ inclined crack.

Table 7.4 Failure load and number of cycles for tested CCNBD samples with inclined notch crack

Sample	Amplitude % SUL	Crack inclination angle, β (°)	Ultimate load (kN)	Number of cycles up to failure
CCNBD-Rp1	10	30	2.79	2241
CCNBD-Rp2	10	30	2.36	1897
CCNBD-Rp3	10	30	2.72	2234
Average			2.62	2124
CCNBD-Rp1	20	30	2.45	1630
CCNBD-Rp2	20	30	3.10	2472
CCNBD-Rp3	20	30	2.51	1698
Average			2.69	1933
CCNBD-Rp1	30	30	3.16	2306
CCNBD-Rp2	30	30	3.39	2541
CCNBD-Rp3	30	30	3.50	2685
Average			3.35	2510
CCNBD-Rp1	10	45	2.65	2232
CCNBD-Rp2	10	45	3.76	3305
CCNBD-Rp3	10	45	3.45	2686
Average			3.28	2741
CCNBD-Rp1	20	45	3.59	2973
CCNBD-Rp2	20	45	2.07	1084
CCNBD-Rp3	20	45	2.58	1800
Average			2.75	1952
CCNBD-Rp1	30	45	2.89	1872
CCNBD-Rp2	30	45	2.41	1308
CCNBD-Rp3	30	45	3.51	2685
Average			2.94	1955
CCNBD-Rp1	10	60	2.41	1900
CCNBD-Rp2	10	60	1.75	1030
CCNBD-Rp3	10	60	2.54	2074
Average			2.23	1668
CCNBD-Rp1	20	60	2.16	1274
CCNBD-Rp2	20	60	2.71	1963
CCNBD-Rp3	20	60	2.13	1253

Average			2.33	1496
CCNBD-Rp1	30	60	2.93	1874
CCNBD-Rp2	30	60	3.76	2886
CCNBD-Rp3	30	60	2.61	1848
Average			3.10	2202

As was previously observed in the static tests, wing cracks, which are tension cracks, initiated at (or near) the tips of the notch crack and propagated parallel to the diametral compressive loading axis. The wing crack initiation position differs for different notch crack inclination angles. Three different wing crack initiation positions were observed. Figure 7.27 shows these positions: at the tip, near the tip and near middle of the notch crack.

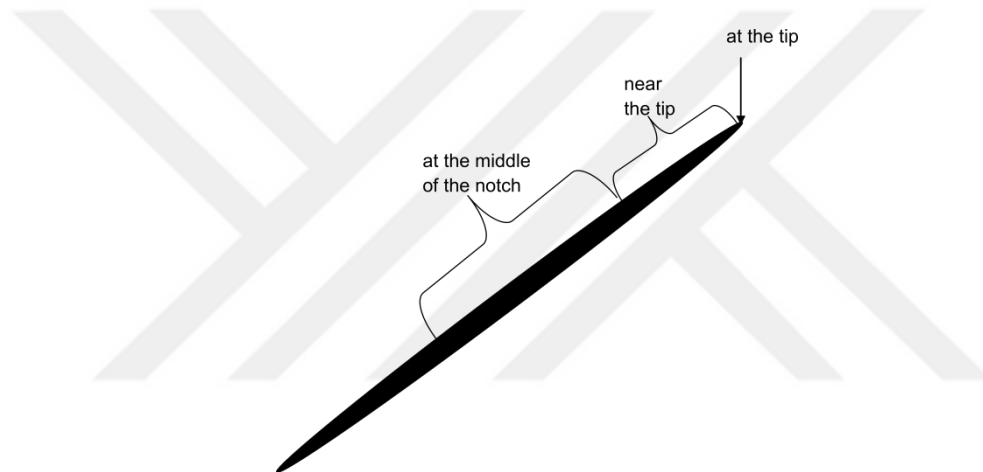


Figure 7.27 Wing crack initiation positions

According to initiation position, wing cracks can be classified as upper external wing cracks, lower external wing cracks and fatigue cracks (see Figure 7.28). Upper and lower external wing cracks were always observed for all the specimens and most of them propagated to the top and bottom edges of the disc specimens at an angle to the crack plane, while fatigue cracks did not propagate along a typical path, as some of them propagated parallel to the notch crack plane and some of them propagated at an angle to the notch crack plane. Moreover, because the brittle fracture evaluation process under load control tests took place fast, it was not possible to capture crack initiation and propagation stages separately. Thus, it is hard to determine whether external wing cracks initiated and propagated before fatigue cracks. It has been believed for static loading conditions that increasing mode II loading of a sample meant that a wing crack is initiated first. Even though the wing fractures, which are tensile by nature, are initiated, with increasing shear load a secondary fracture is instigated (Barker, 2004). However, mixed-mode fracturing is evident on the microscale and different mechanisms contribute to crack propagation under cyclic loading.

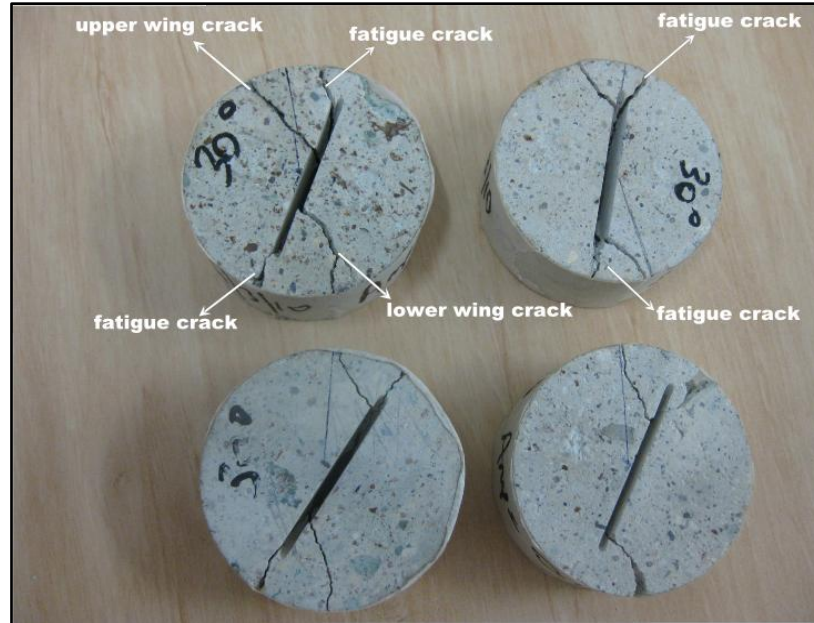
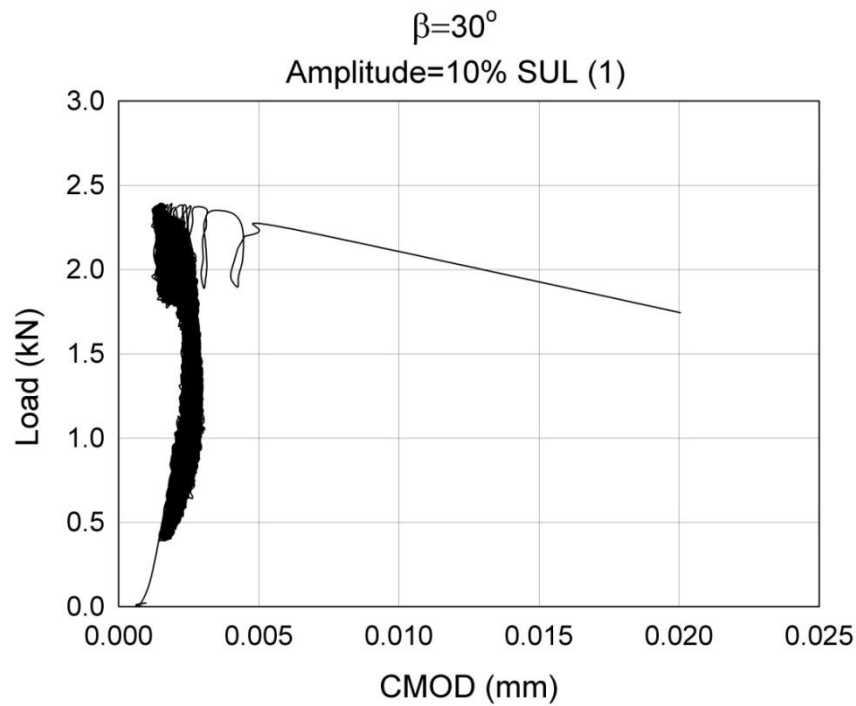


Figure 7.28 $\beta = 30^\circ$ inclined notch and newly formed crack patterns on CCNBD specimens tested under diametral compressive cyclic loading

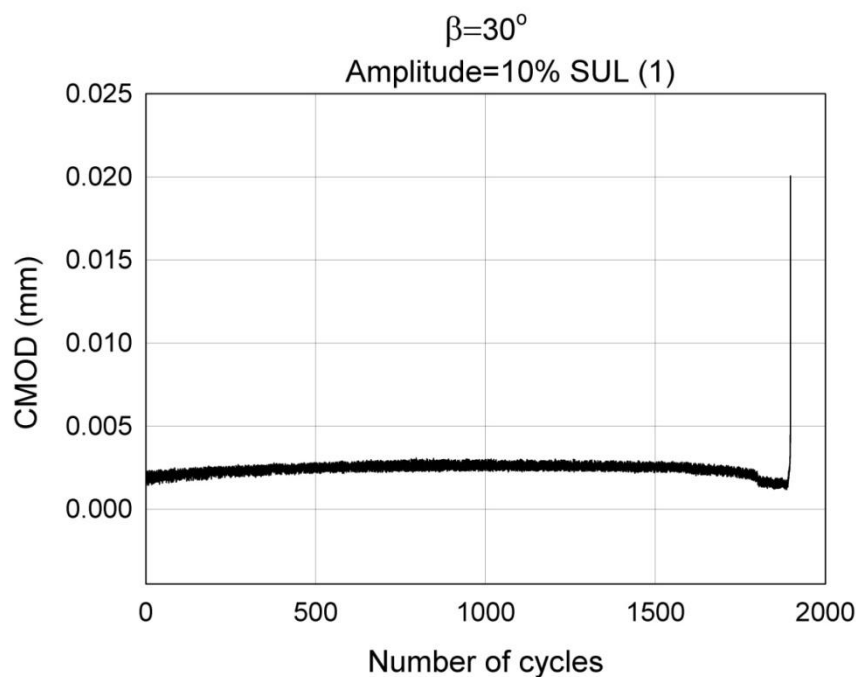
However, when the geometrical distributions of cracks in the CCNBD samples are different, the coalescence modes of the fractures in rock should be different under static or cyclic loading. Certainly, the total deformation or irreversible deformation is different too. The general finding from the inclined notch crack tests is that any strain softening in the postpeak region in load–displacement plots as was observed in mode I cyclic loading tests was not observed with mixed-mode cyclic loading tests. Figure 7.29 shows the load–CMOD and CMOD–number of cycles plots for $\beta = 30^\circ$ inclined CCNBD specimens under diametral compressive cyclic loading with 0.45 kN, 10% of SUL amplitude. Very interesting results were obtained with load–CMOD plots as a result of the irreversible strain accumulation with decreasing crack mouth opening rate. For instance, normal stress perpendicular to the crack plane caused the crack to open with a very small constant strain accumulation under increasing cyclic loading then the crack started to open up to failure with a fast rate (Figure 7.29b).

There are several possible mechanisms relating to the initiation of cracks, including: crack tip bridging, crack branching, the fragmentation of individual grains and the spalling of grain assemblages. It is also possible that the start of opening in the CMOD–load plot shown in Figure 7.29a would show the start of a fatigue crack initiation along the mouth of the notch crack causing loosening and opening of the notch crack. In contrast, there is a clear accumulation of axial irreversible strain with increasing number of cycles (Figure 7.29c) while there is almost no

irreversible strain accumulation with CMOD readings along the crack plane. It is believed that compaction would involve both the development of cracks along grain boundaries through the weaker segment of the cement matrix and rotation or movement of the intact grains into a tighter alignment parallel to the direction of diametral compressive loading. Those possible mechanisms are discussed in detail in the following sections using images captured by SEM.



(a)



(b)

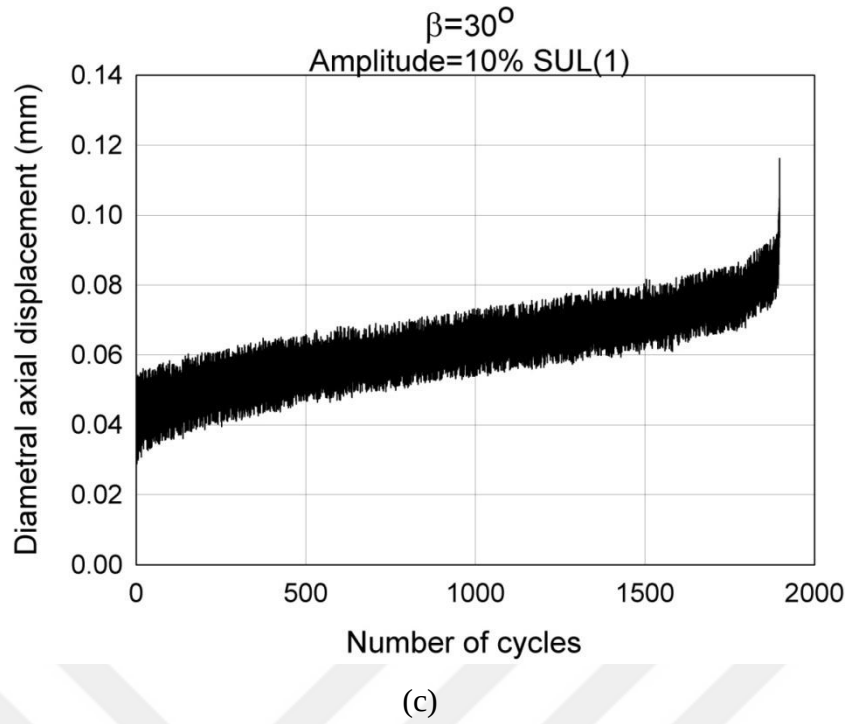
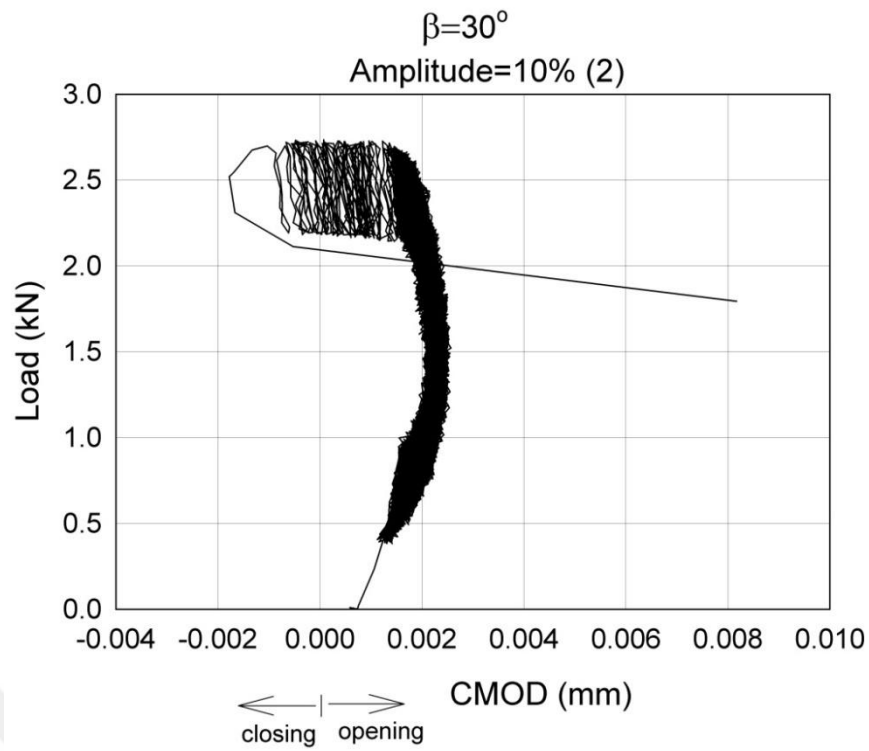
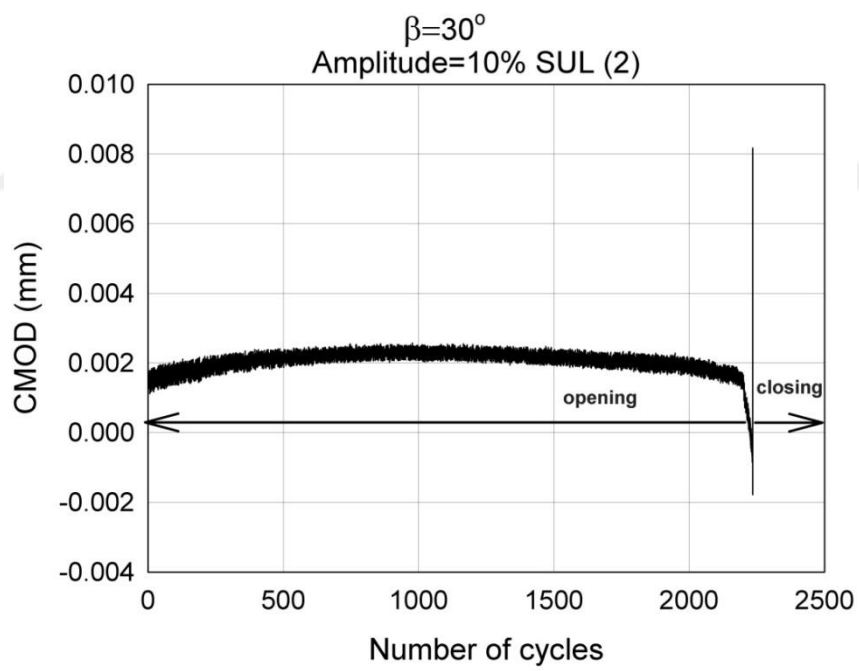


Figure 7.29 Behaviour of $\beta = 30^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 0.45 kN amplitude

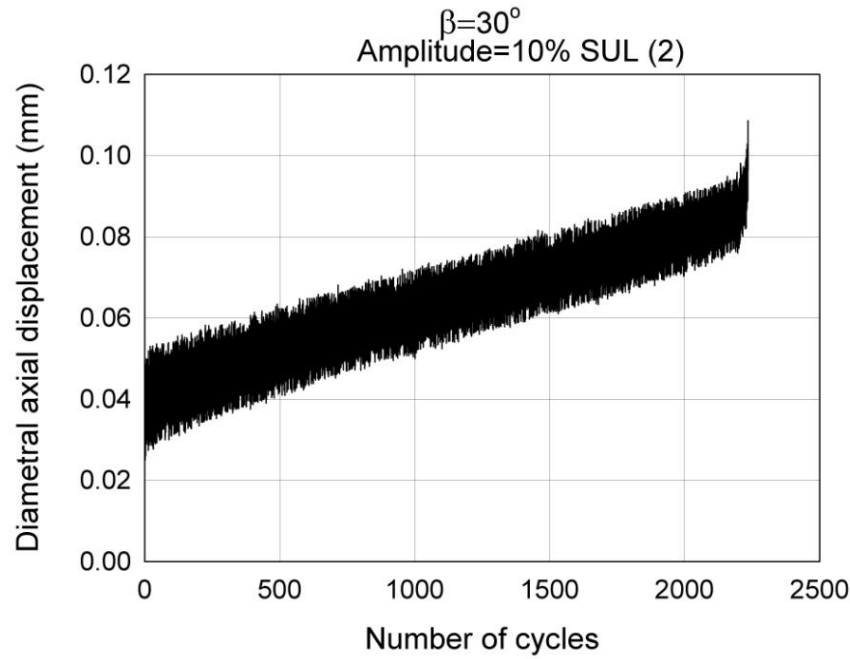
Another set of plots for $\beta = 30^\circ$ inclined CCNBD specimens under diametral compressive cyclic loading is shown in Figures 7.30a, 7.30b and 7.30c. The amount of accumulated irreversible strain measured along the crack mouth is very small, 0.002 mm, and almost constant up to the start of unstable crack propagation. The fracturing process of this notch crack is different from the previous one shown in Figure 7.29, because the notch crack first started to open with very small accumulated strain, 0.002 mm, and then started to close up to failure. Almost the same irreversible diametral axial displacement, 0.11 mm, was obtained as in the previous example, shown in Figure 7.29c. However, the sample shown in Figure 7.30 resisted more loading cycles with almost the same amount of strain as the sample shown in Figure 7.29.



(a)



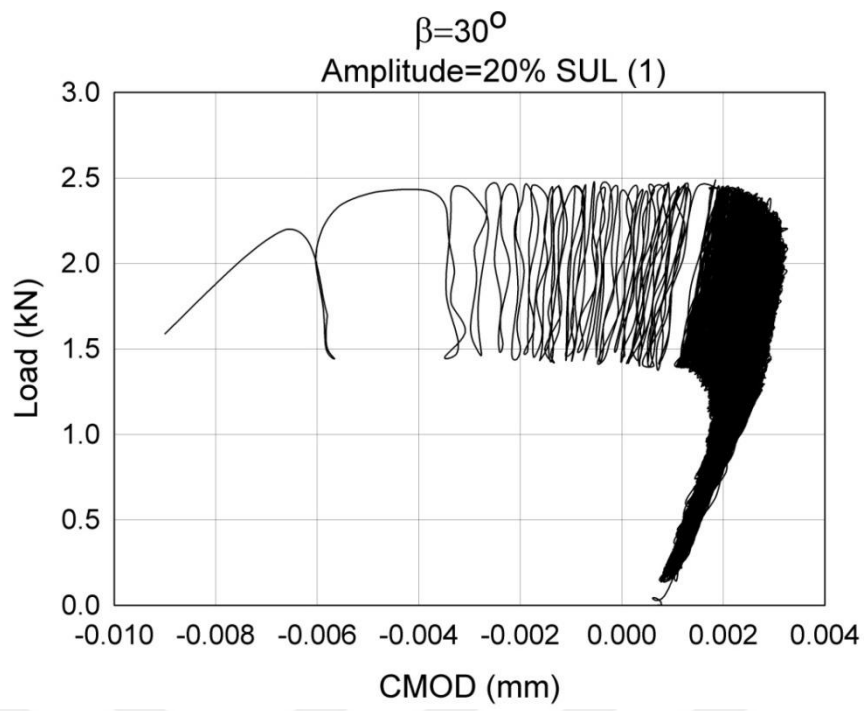
(b)



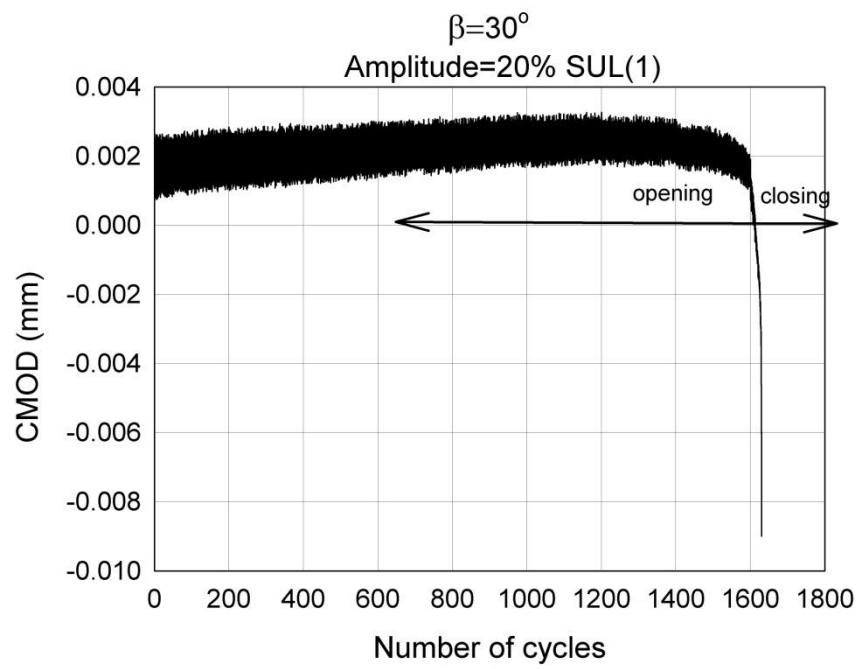
(c)

Figure 7.30 Behaviour of $\beta = 30^\circ$ inclined notch crack in a CCNBD specimen tested under diametral compressive cyclic loading with 0.45 kN amplitude

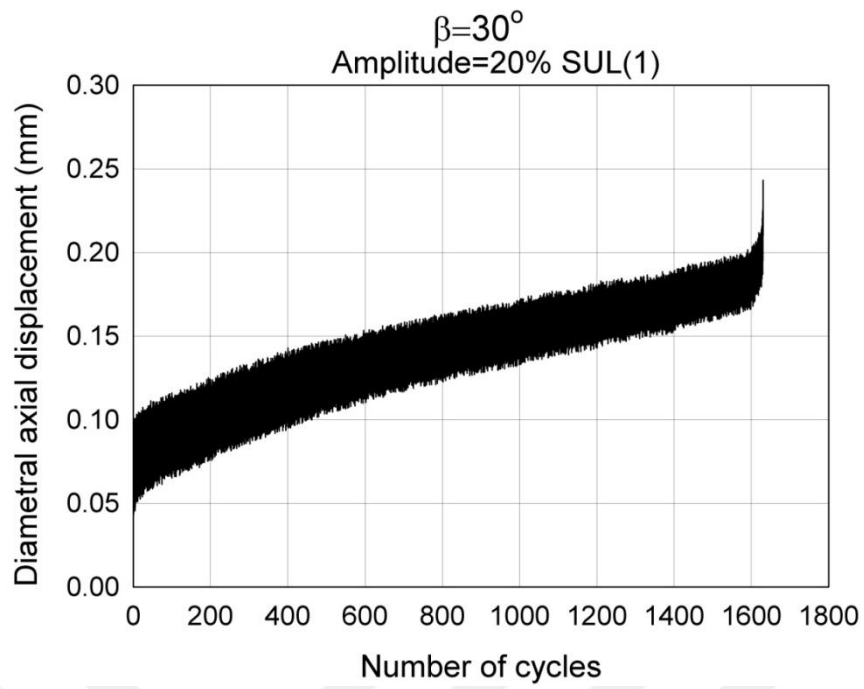
Following the $\beta = 30^\circ$ inclined crack with 0.45 kN amplitude tests, various amplitude cyclic loading tests were carried out with same crack inclination angle. Three repeats were done for each amplitude cyclic test and one of the test results is shown in Figure 7.31 and Figure 7.32. The other plots can be found in Appendixes?. With increasing amplitude values, no specific failure load decrease or increase was seen. As was pointed out before, the notch crack did not show typical behaviour because it opened first and then closed up to failure in some tests or it closed first then opened up to failure. However, it is obvious that an accumulation of plastic deformation always took place in rock material because of rock fatigue, whether the pre-existing crack is inclined or not. It was understood that the crack behaviour depends on the FPZ in front of the tip of the notch crack. A detailed analysis of damage mechanisms in the FPZ zone in front of the tip of the notch crack is given in following sections.



(a)

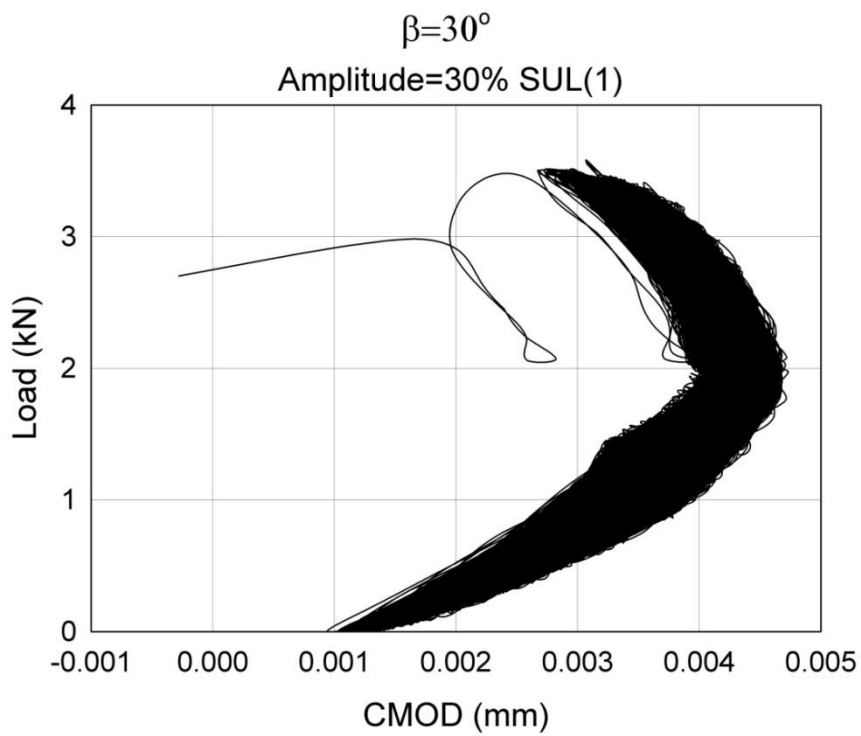


(b)

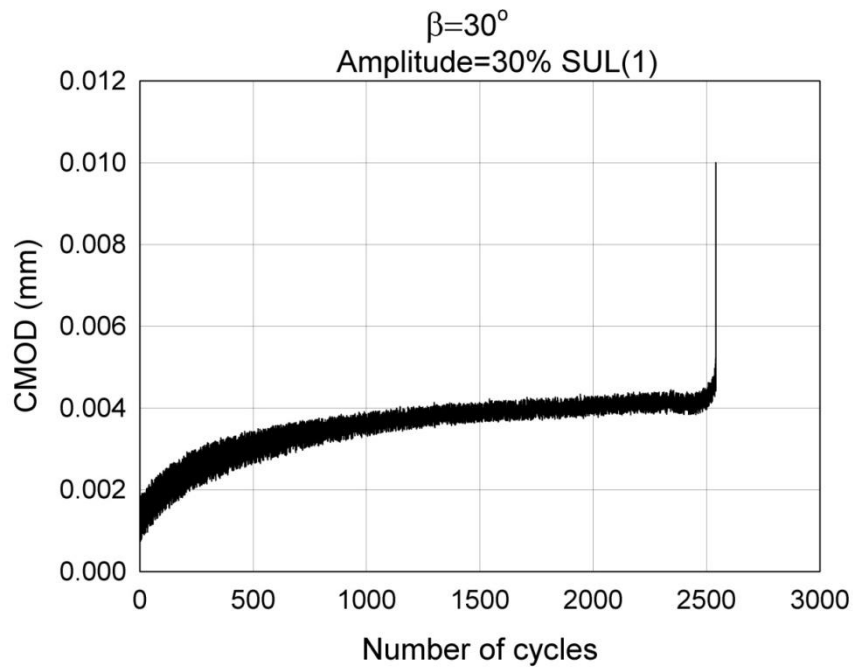


(c)

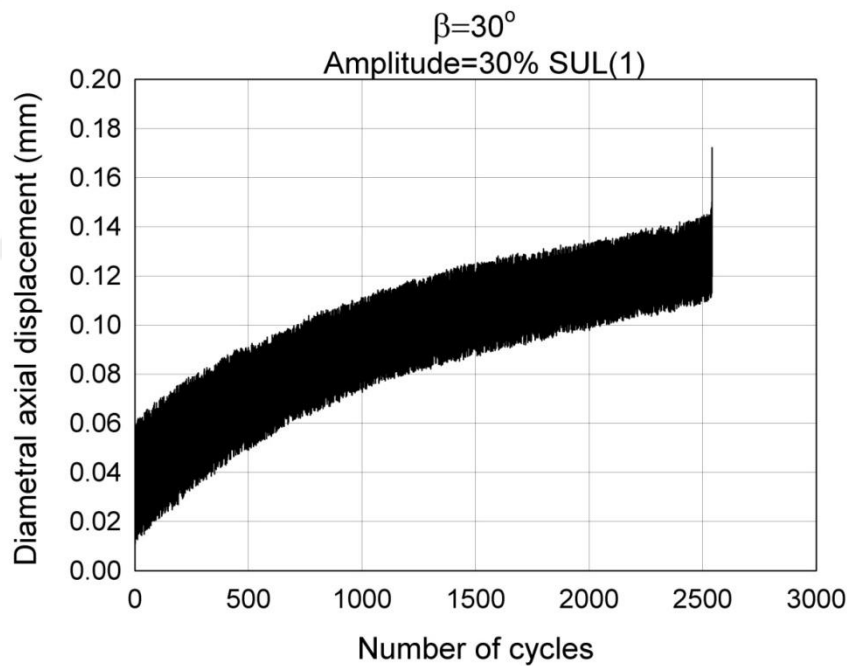
Figure 7.31 Behaviour of $\beta = 30^\circ$ inclined notch crack in a CCNBD specimen tested under diametral compressive cyclic loading with 0.9 kN amplitude



(a)



(b)



(c)

Figure 7.32 Behaviour of $\beta = 30^\circ$ inclined notch crack in a CCNBD specimen tested under diametral compressive cyclic loading with 1.35 kN amplitude

The tested $\beta = 45^\circ$ inclined notch crack samples are shown in Figure 7.33. Results from the diametral compressive cyclic tests showed that the development of wing cracks and fatigue cracks are similar to that seen in the $\beta = 30^\circ$ inclined crack under cyclic loading test. However, fatigue cracks initiate closer to the centre of the notch crack at this angle instead of initiating close to the tip

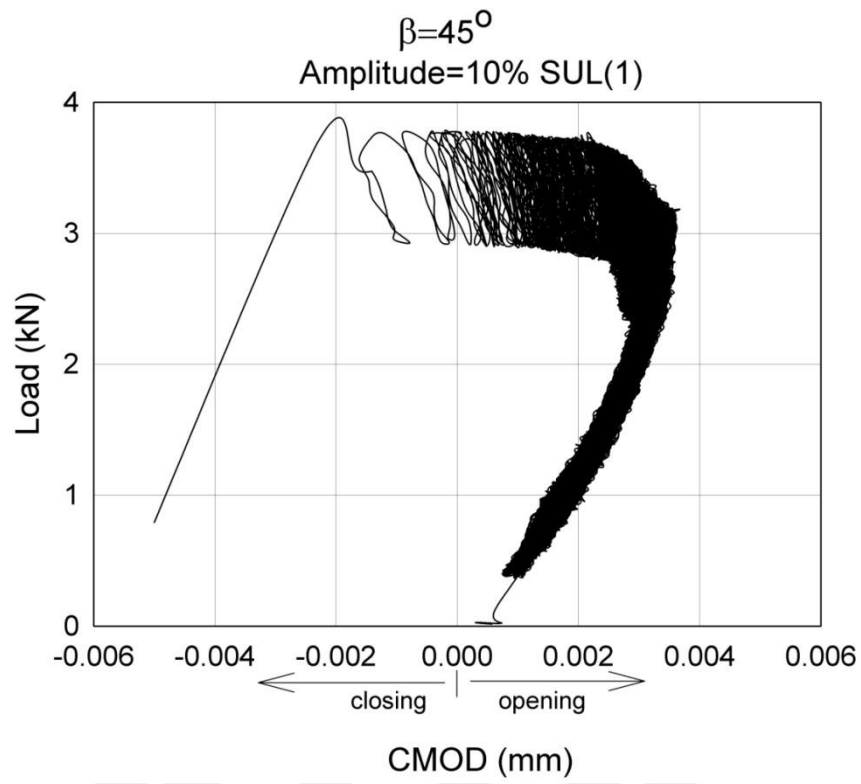
of the notch crack and almost parallel to the notch crack plane. It is possible to say that the extra fatigue cracks may be caused by the tensile crack driving forces along the notch crack mouth. However, it is clearly always possible for extra cracks to occur around the tip of the notch crack due to the effect of rock fatigue. Those cracks can interact and coalesce with one another to produce continuous plastic strain accumulation to form more crushed/small particles than in statically failed specimens.



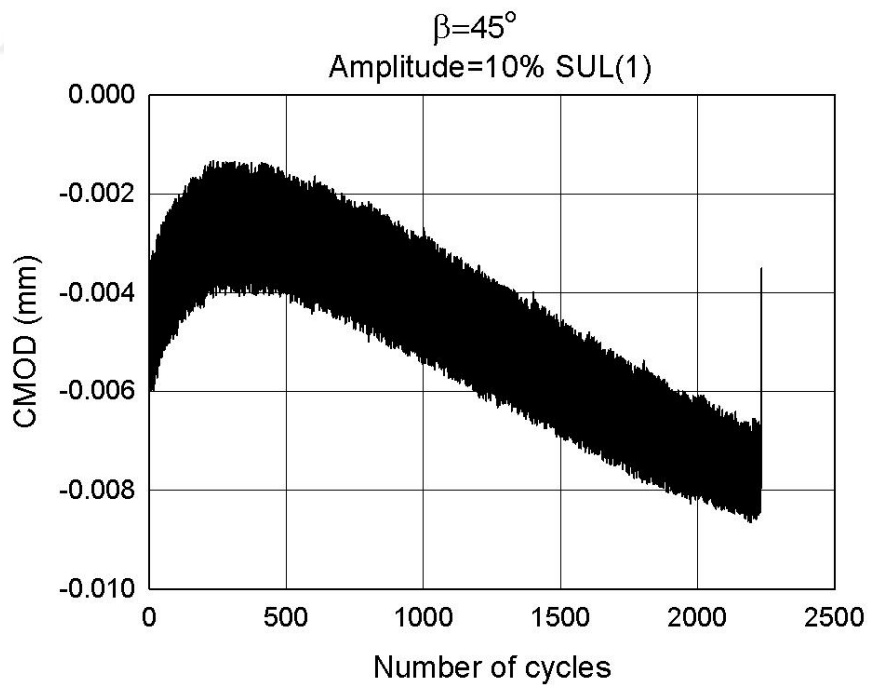
Figure 7.33 $\beta = 45^\circ$ inclined notch and newly formed crack patterns on CCNBD specimens tested under diametral compressive cyclic loading

One of the most interesting outcomes in this research was obtained with $\beta = 45^\circ$ inclined notch crack behaviour under diametral cyclic loading. $\beta = 45^\circ$ inclined notch cracks were found to open up to failure in all three cyclic loading test repeats. However, the $\beta = 45^\circ$ crack inclination angle had been determined in Chapter 5 as an angle where pre-existing notch cracks closed up to failure. This behaviour was also obtained with some $\beta = 30^\circ$ and $\beta = 60^\circ$ inclined cracks too. Compared with the static damage mechanism for $\beta = 45^\circ$ inclined notch crack behaviour, there should be a different fatigue damage mechanism in front of the tip of the $\beta = 45^\circ$ inclined notch crack under cyclic loading. The details are given in following section.

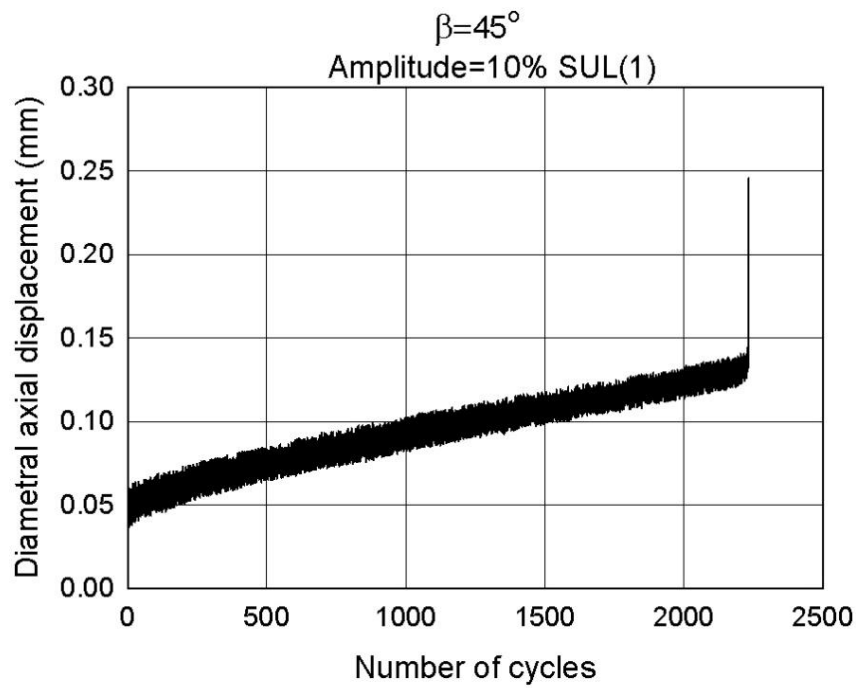
Load–CMOD, CMOD–number of cycles and axial displacement–number of cycles plots for the $\beta = 45^\circ$ inclined notch crack under various amplitudes of cyclic loading are shown in Figure 7.34, Figure 7.35 and Figure 7.36. Other plots of test repeats are given in Appendixes ?



(a)

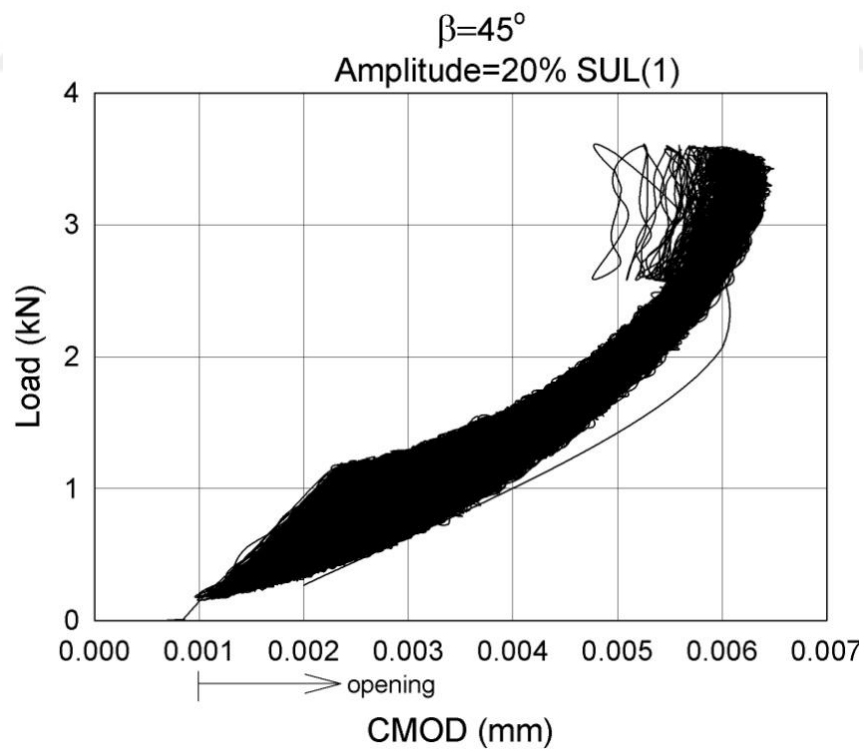


(b)

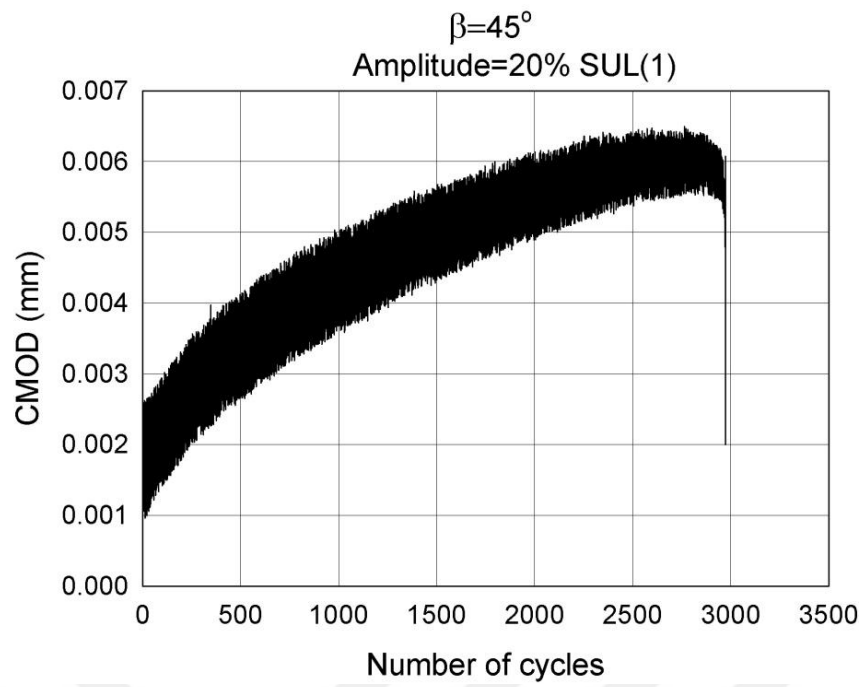


(c)

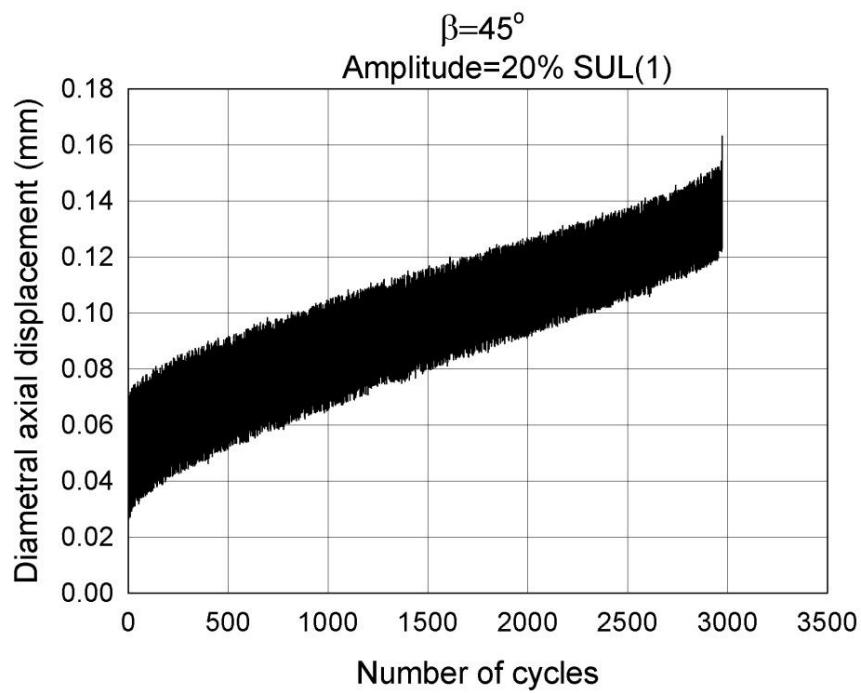
Figure 7.34 Behaviour of $\beta = 45^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 0.45 kN amplitude



(a)

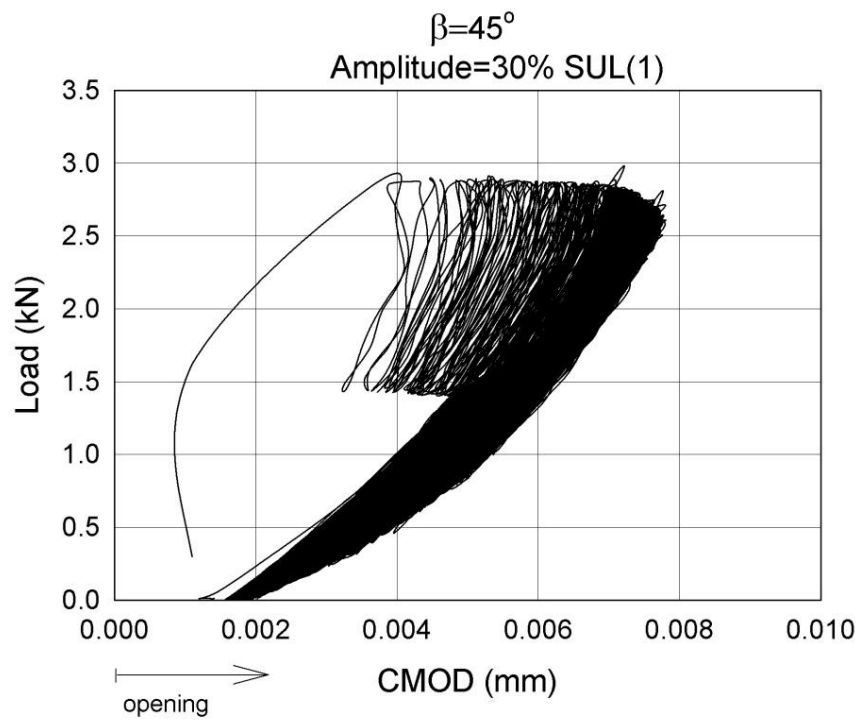


(b)

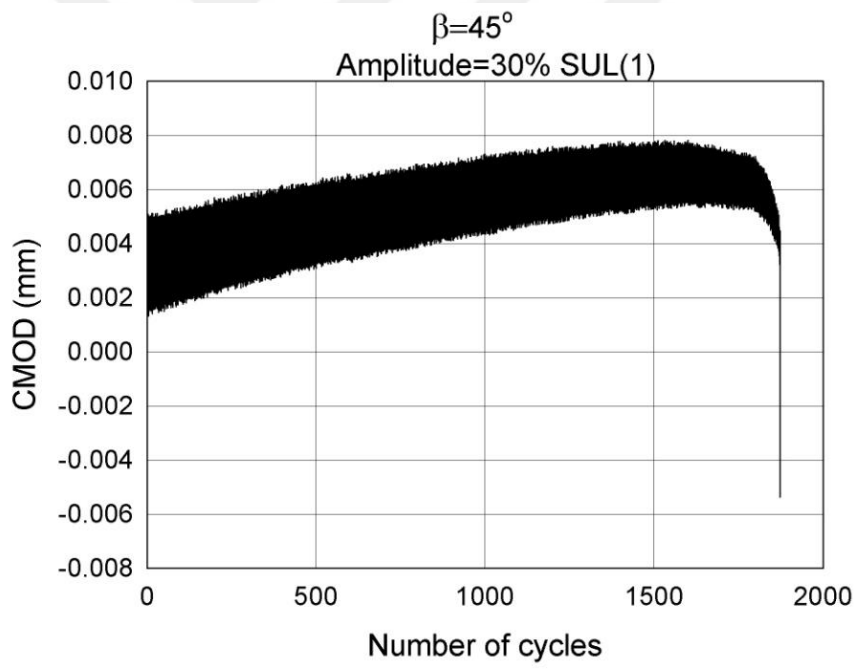


(c)

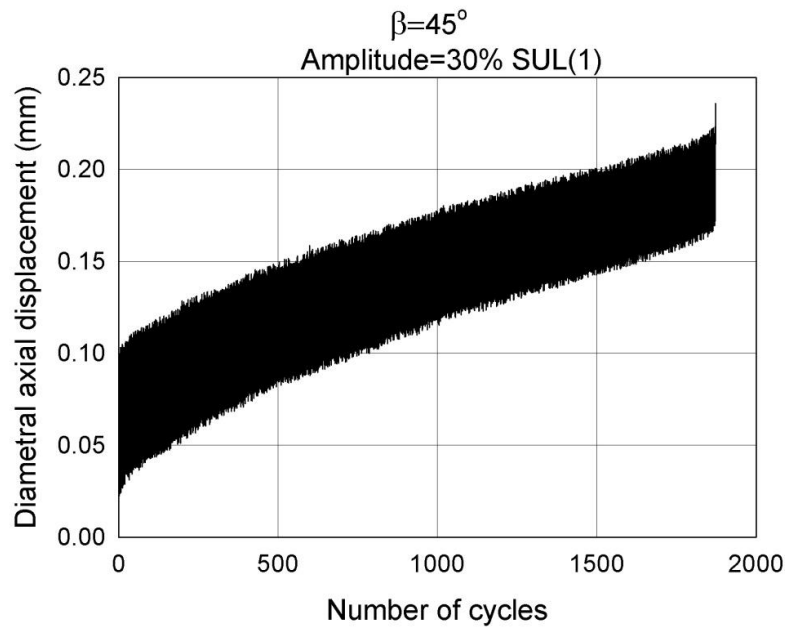
Figure 7.35 Behaviour of $\beta = 45^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 0.9 kN amplitude



(a)



(b)



(c)

Figure 7.36 Behaviour of $\beta = 45^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 1.35 kN amplitude

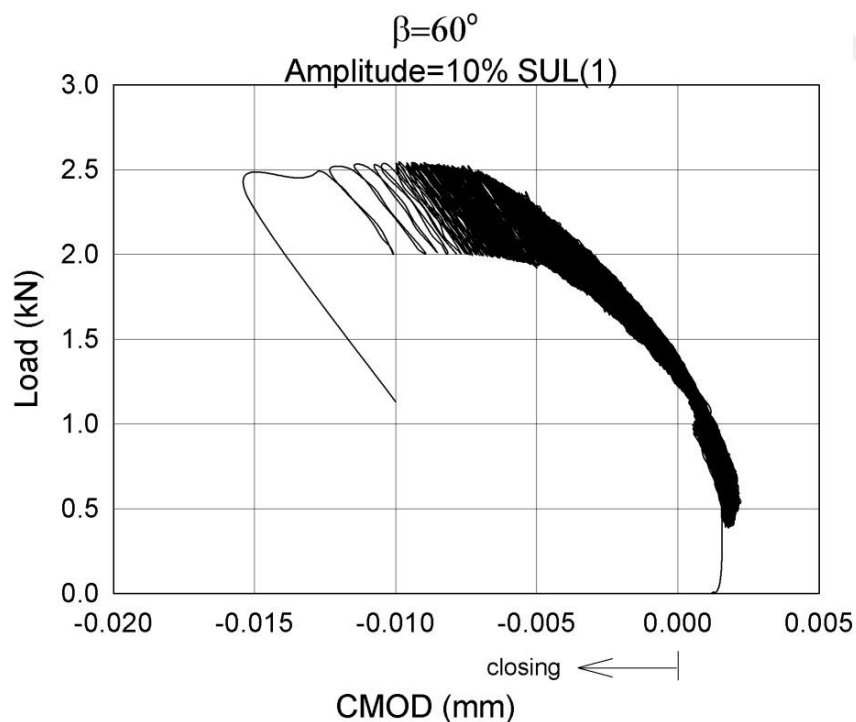
The tested $\beta = 60^\circ$ inclined notch crack samples are shown in Figure 7.37. Results from this test were similar to the wing and fatigue crack trajectories obtained with $\beta = 30^\circ$, $\beta = 45^\circ$ and $\beta = 60^\circ$ inclined notch cracks. However, fatigue crack propagation at the tips of the $\beta = 60^\circ$ inclined notch crack is more parallel to the notch crack plane compared with those at $\beta = 30^\circ$ and $\beta = 45^\circ$ crack trajectories.



Figure 7.37 $\beta = 60^\circ$ inclined notch and newly formed crack patterns on CCNBD specimens tested under diametral compressive cyclic loading

Load–CMOD, CMOD–number of cycles and axial displacement–number of cycles plots for $\beta = 60^\circ$ inclined notch crack under various amplitudes of cyclic loading are shown in Figure 7.38, Figure 7.39 and Figure 7.40. Other plots of test repeats are given in Appendixes ? Similarly to the $\beta = 45^\circ$ inclined crack, $\beta = 60^\circ$ crack inclination angle was obviously determined in Chapter 5 as an angle where the pre-existing notch cracks closed up to failure. However, some of the $\beta = 60^\circ$ inclined notch cracks were found to open up to failure in cyclic loading tests.

Test results for inclined notch cracks under cyclic loading show that the typical CMOD response to cyclic loading is totally different from the response to static loading, while the axial displacement response under cyclic loading is similar to the response under static loading. Once this point was reached, irreversible strain accumulation for CMOD under cyclic loading is not as much as that under static loading. In other words, notch crack failure occurs at a very small irreversible CMOD value while axial displacement is continuously increasing. These results again clearly show that different damage mechanisms take place in front of the inclined notch crack due to rock fatigue. Additionally, it is hard to understand and explain how the $\beta = 60^\circ$ inclined notch crack can open up to failure under cyclic loading while this angle is definitely determined as a crack closing angle under static loading.



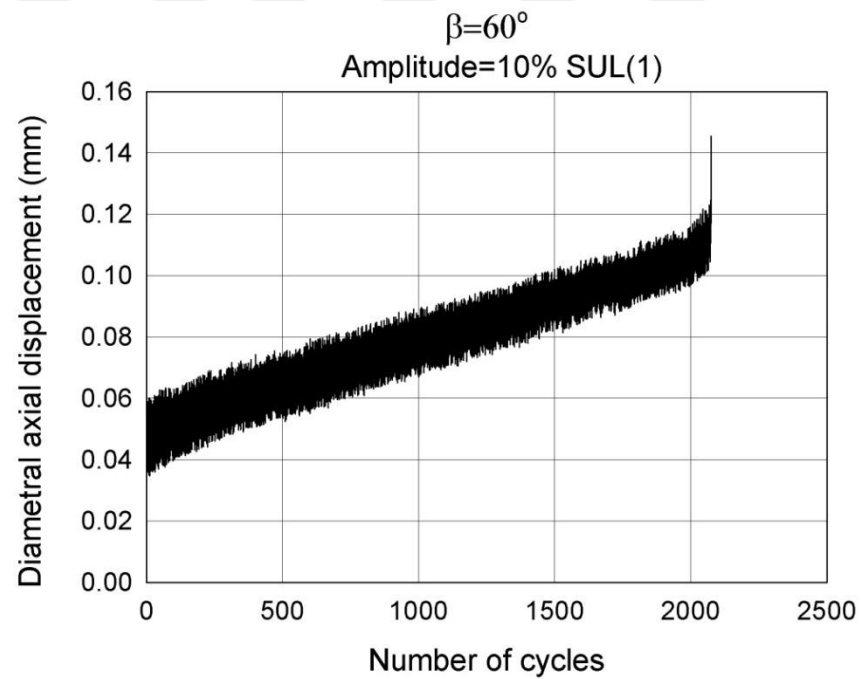
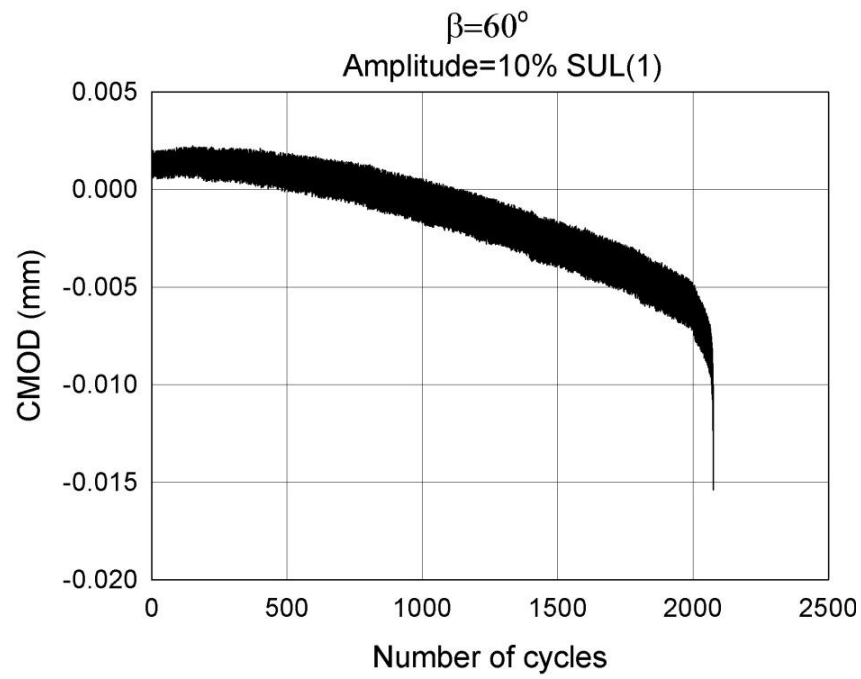
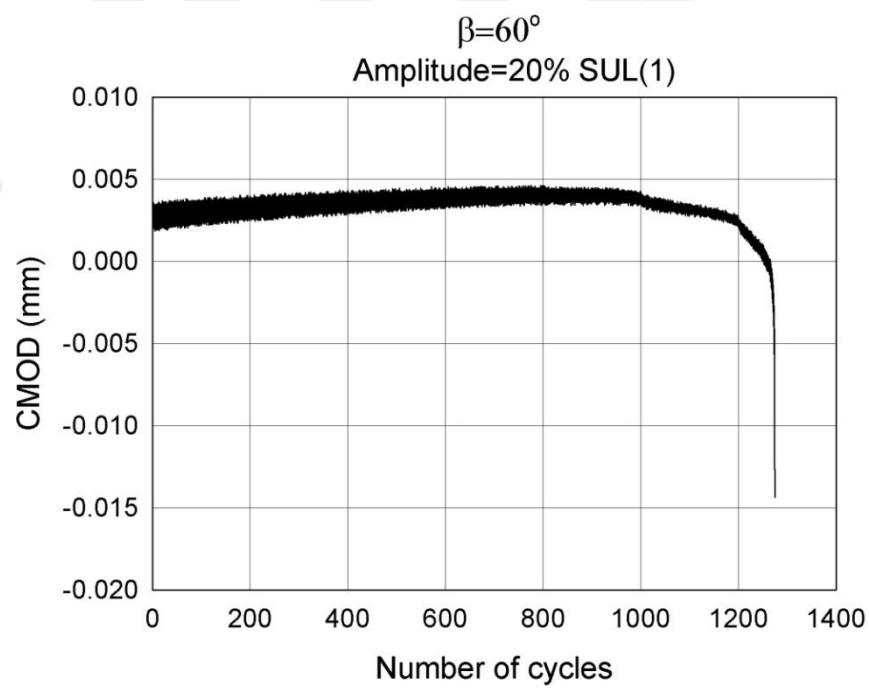
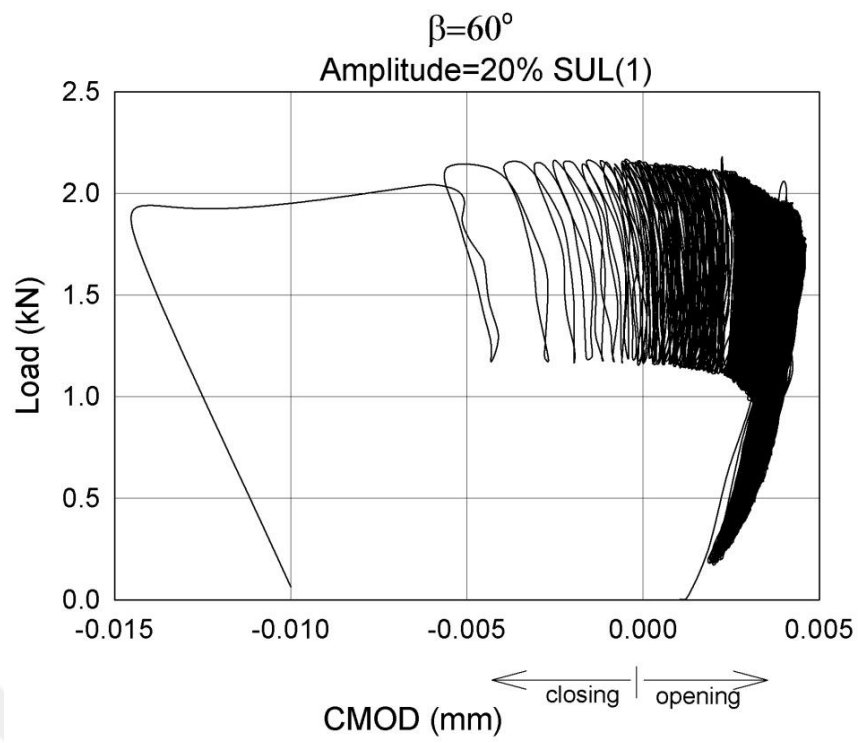


Figure 7.38 Behaviour of $\beta = 60^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 0.45 kN amplitude



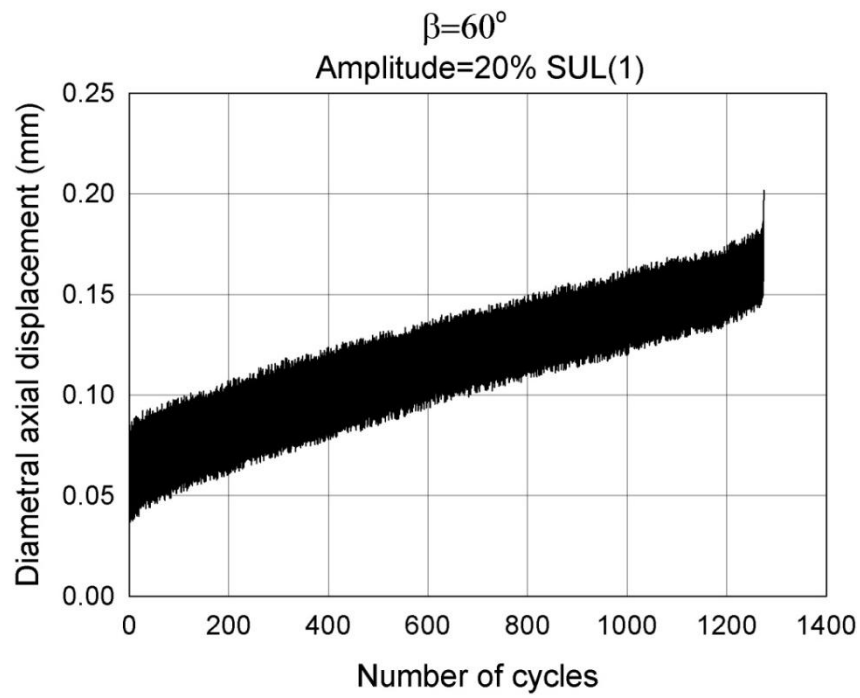
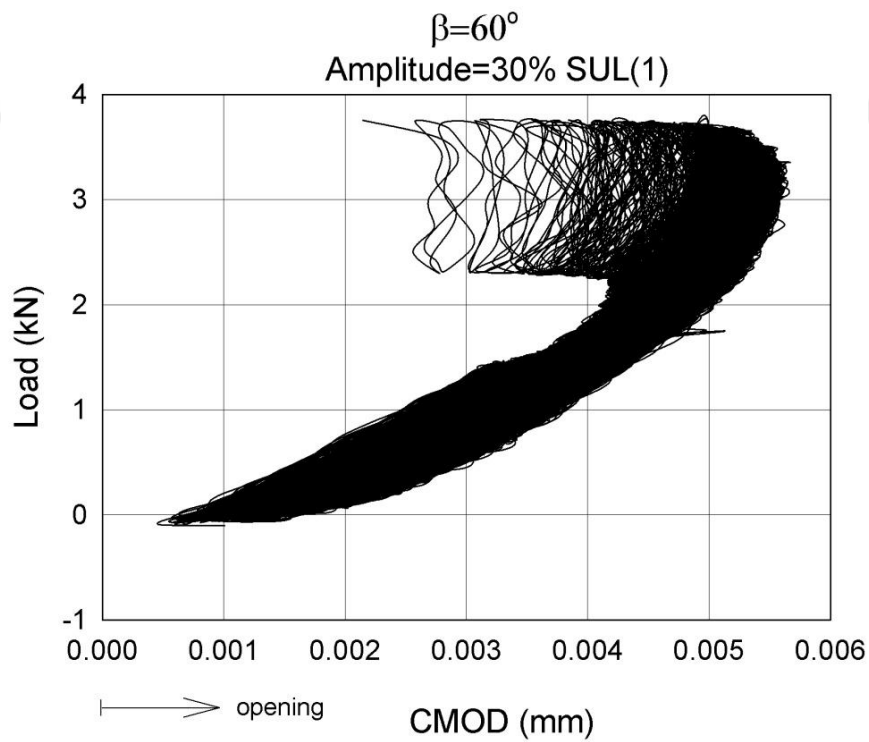


Figure 7.39 Behaviour of $\beta = 60^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 0.9 kN amplitude



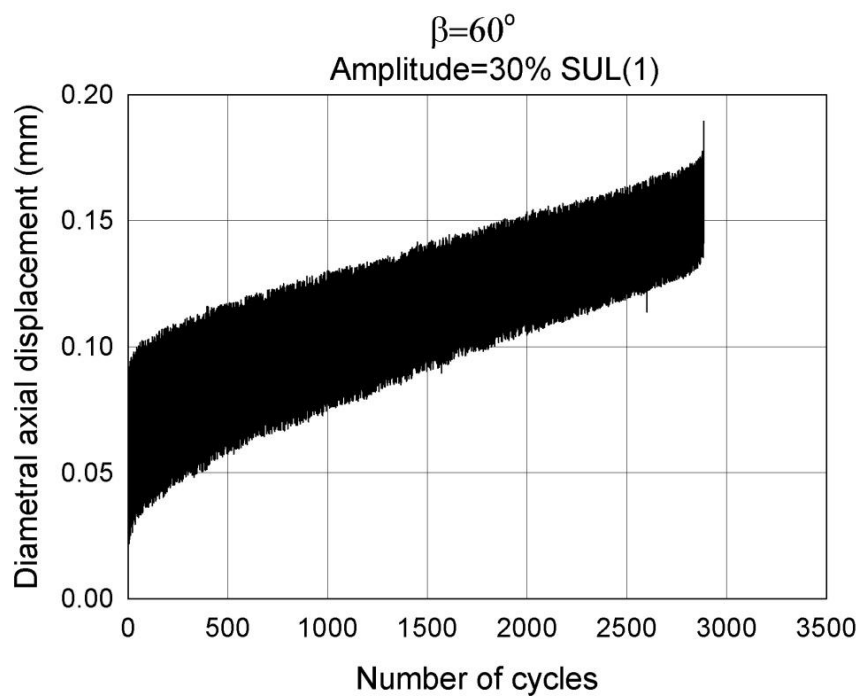
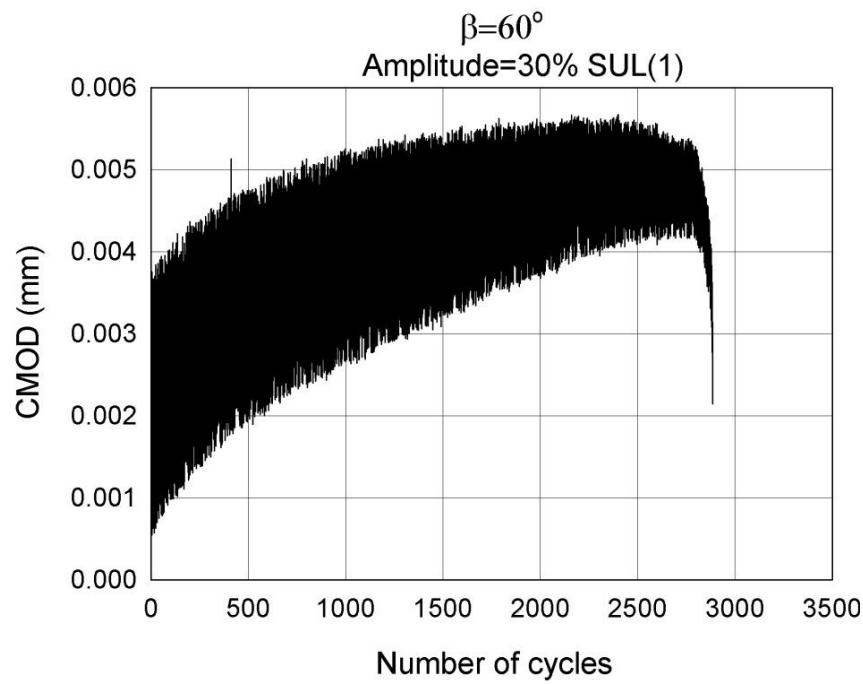


Figure 7.40 Behaviour of $\beta = 60^\circ$ inclined notch crack in CCNBD specimen tested under diametral compressive cyclic loading with 1.35 kN amplitude

7.4 The Mechanism of Rock Fatigue Damage in Front of Chevron Notched Crack

Simmons and Richter (1976) define the word microcrack as:

“An opening that occurs in rocks and has one or two dimensions smaller than the third. For flat microcracks, one dimension is much less than the other two and the width to length ratio, termed crack aspect ratio, must be less than 10^{-2} and is typically 10^{-3} to 10^{-5} . The length typically is of the order of $100\ \mu\text{m}$ or less.”

They subdivide microcracks into grain boundary cracks (associated and perhaps coincident with the grain boundary), intragranular or intracrystalline cracks (lying totally within the grain) and intergranular or intercrystalline cracks (extending from a grain boundary crossing into one or more other grains). The term transgranular crack is sometimes used for cracks running across a grain from grain boundary to grain boundary. However, cleavage cracks are an important enough subset of intracrystalline cracks to be considered separately. They are separations along cleavage planes in minerals and, as such, are petrographically useful. They often occur in parallel sets of various lengths within one grain. More than one cleavage plane within a grain may be cracked. Unless the mineral has been kinked, the cleavage crack walls are extremely sharp and virtually parallel except where they gradually merge with uncracked material.

Under the experimental and natural conditions where cracks are known to exist, mechanical stresses can generate cracks through at least six mechanisms. These include microcracks produced as a result of (1) twin interactions with grain boundaries and other twins; (2) release of stored strain energy associated with kink bands and deformation; (3) cleavage separations; (4) stress concentrations in the neighbourhood of grain boundaries, intracrystalline cavities and crack tips; (5) mismatches in elastic compliances of neighbouring grains; and (6) grain translations and rotations.

Generally, the crack will tend to follow the local maximum stress trajectory under monotonic loading. The presence of cleavage planes, pores, other cracks and inclusions will affect the crack propagation path by modifying the stress field in its vicinity or by introducing anisotropy in local cohesive strength. This fracture mechanics principle is one of the major factors in determining the process of fatigue crack propagation under diametral compressive cyclic loading in this research.

Because the CMOD was found to be different for vertical and aligned notch cracks, fatigue mechanism in front of the tip of a crack is explained in this research separately for (1) vertically aligned notch cracks (mode I cracks) and (2) cracks aligned at an angle (mixed-mode I–II). Mode I notch cracks are used for vertical cracks and mixed-mode notch cracks are used for inclined cracks.

It was found from the experimental outputs that there is effective accumulated plastic deformation in front of the notch crack prior to failure under cyclic loading. According to NEFM, a significant FPZ is expected in front of the tip of the notch crack with high plastic deformation before failure of our disc samples. One important output of our experimental study on mode I notch cracks was obtaining a clear tensile softening zone after the postpeak region in all CMOD–load graphs. This tension-softening behaviour is responsible for the development of the FPZ zone in front of and around the crack tip. The cohesive crack model is able to describe materials that exhibit strain-softening behaviour (Dugdale, 1960). It should be mentioned here that this tension-softening behaviour was not seen in the static loading tests; however, a clear tension-softening curve was obtained in the postpeak region in all CMOD–load graphs of cyclic loading tests even with various load amplitudes. One tension-softening curve is shown in Figure 7.41. Because tension-softening research in literature was mostly done under quasi-static loading (controlled tension tests) conditions, the tensile softening phenomenon needs to be extended for rock fatigue research under tensile cyclic loading conditions. For example, in concrete, microcrack branching, aggregate bridging and other micromechanisms have been identified as being responsible for the tension-softening process (Horii *et al.*, 1992; Li & Maalej, 1996).

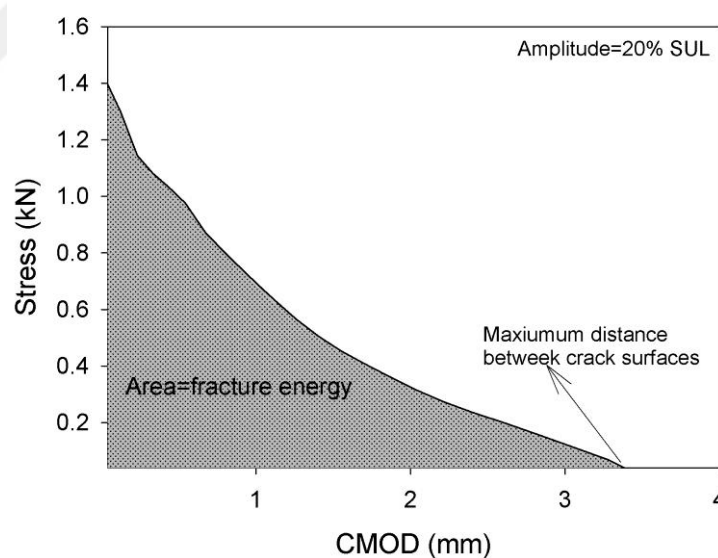


Figure 7.41 Tension-softening curve

However, the microcracking damage process that causes the tension-softening behaviour under cyclic loading needs to be determined. The cohesive crack model proposed in literature may not be able to explain the observed tension-softening behaviour under cyclic loading in this research. There are some limitations among cohesive crack formulations: the crack tip faces close smoothly (the stress intensity factor K_I vanishes at the crack tip in mode I propagation) and the fracture

process zone is of negligible thickness. The closing stresses in the fracture process zone are constant in Dugdale's model, while the size of this zone is constant and small in comparison with the length of the main crack in Barenblatt's model (Barenblatt, 1962; Dugdale, 1960 and Hillerborg, 1983). All existing models in the literature for the FPZ in front of the notch crack in brittle materials were proposed for the dynamic loading situation by using some assumptions, e.g., the rock is assumed linear elastic up to failure. However, cyclic loading is not a kind of impact loading and, thus, the rock material around the notch crack cannot be assumed to be a linear elastic material. Therefore, direct microscopic imaging of the damaged zone in front of the tip of the chevron crack was used instead of modelling the FPZ zone by using or extending the existing quasi-static models to explain strain softening and damage around the tip of the notch crack under cyclic loading. SEM studies were therefore conducted on the tested specimens to observe the nature of cracking directly.

One of the most important results of this study is that fracture toughness can be reduced under repetitive (cyclic) loading. This means that crack propagation can take place under lower than expected ultimate loads compared with static loading. The mode I stress intensity factor K_I at the crack tip is known to control the crack growth. The stress corrosion lower limit is K_o , and the fracture toughness is K_{IC} . When $K_I > K_{IC}$, the crack grows catastrophically at approximately the speed of sound, when $K_I < K_o$, the crack does not grow and when $K_o < K_I < K_{IC}$, the crack grows at a certain velocity with the stress intensity factor K_I . However, it is shown in this research that macrocrack propagation causing failure happened with lower stress intensity values (K_I) at the crack tip than the critical stress intensity value (K_{IC}). This result goes against the classical theory, which predicts that there will be no crack growth as long as $K_I < K_{IC}$. This phenomenon is known as 'subcritical crack growth' (Atkinson, 1984; Atkinson & Meredith, 1987). Subcritical crack growth is one of the main explanations for creep damage mechanism in rocks.

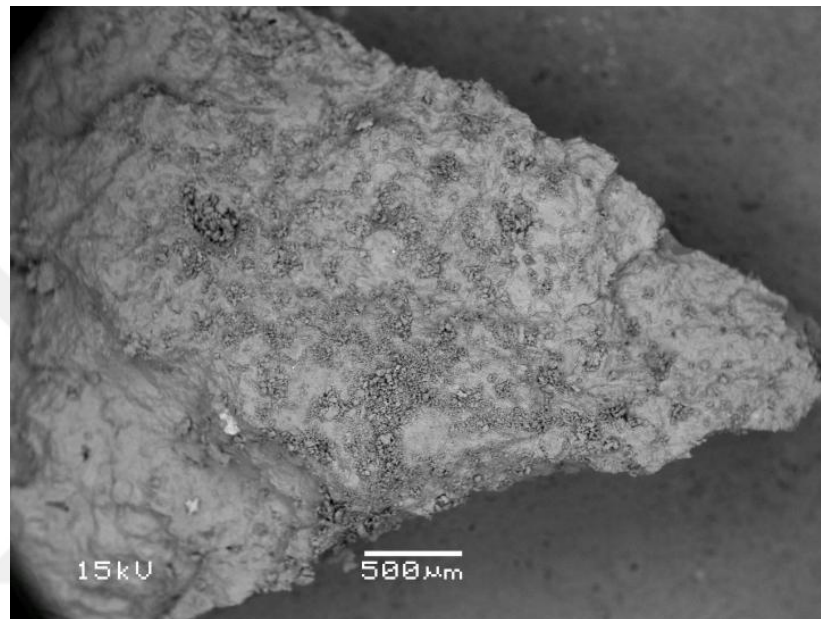
In some fatigue research, damage accumulation in brittle materials under cyclic loading has been explained by the 'creep' mechanism and 'stress corrosion' (Celestino & Bortolucci, 1995; Costin & Holcomb, 1981; Evans, 1976; Xiao *et al.*, 2009). Although different test environments, geometries and loading boundary conditions were used in those past studies, we found in this study similar damage behaviour in front of the notched crack in our disc shape samples as a result of dilatant creep damage behaviour. It was possible to measure the crack growth rate for subcritical crack growth in other studies under controlled stress corrosive environments with edge cracked specimens; however, it is not possible to measure the crack growth rate in our cyclic loading experiments because of the unknown stress corrosive mechanism and the very high-speed unstable

crack propagation. Further, there was not a pre-existing crack for proper monitoring in our three-dimensional sample geometry because chevron notched specimens do not need to be precracked for rock fracture research.

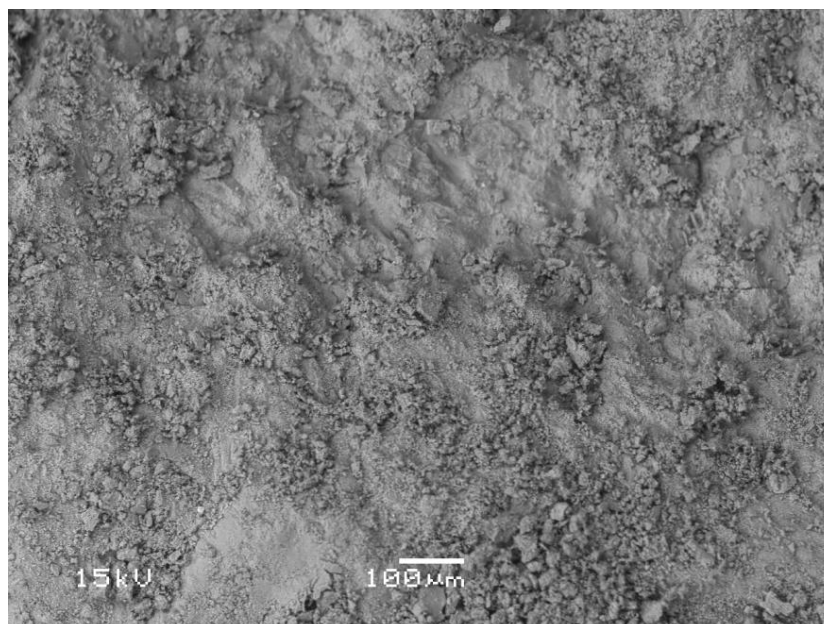
Stress corrosion is the most common mechanism associated with subcritical crack growth in rock. However, it is certainly not the only mechanism by which subcritical crack growth occurs. Another mechanism that can be important, in certain circumstances, is fatigue crack growth. In this study it was shown that the number of cycles to failure decreased when the amplitude of cycles increased. It might be thought that this effect could also be explained by stress corrosion. Stress corrosion and fatigue represent the major mechanisms of subcritical crack growth in rocks (Attkinson, 1987; Costin & Holcomb, 1981). Most of the experimental work on stress corrosion crack growth in rocks and ceramics used methods originally developed for determining fracture toughness in metallic materials. The corrosive environment is water and there is a chemically active environment in those metallic material experiments. However, some mechanisms were accepted as additional mechanisms of subcritical crack growth, such as cyclic fatigue (Costin & Holcomb, 1981; Hsieh & Thomson, 1973; Kim & Mubeen, 1981). In this research, it is believed that fatigue cracks are the dominant mechanisms of subcritical crack propagation. Thus, the main aim here is to discover the possible mechanism of microcracks causing the corrosive environment. Hence, direct observation of the FPZ in front of the tip of the main crack in the tested CCNBD specimens using the SEM is a suitable observation technique for this research.

The characteristics of the fracture surfaces were examined by means of a JEOL JMS-6460 LA SEM. The JEOL JSM-6460 LA is a tungsten low-vacuum analytical scanning electron microscope. In this study, all rock fracture images were obtained under LV chamber pressures, i.e., 1–50 Pa (with adjustable pressure between 10 Pa and 270 Pa). This allows certain samples to be observed uncoated and reduces damage to the specimens caused by the effects of high vacuum. LV mode uses only BSE to observe an image. This is good for our purposes because secondary electron images of the SEM and optical images could not be used to distinguish grain boundaries clearly, it is not certain if a microcrack along a grain boundary extends the entire length of the boundary. In the present study, we used the tested CCNBD specimens and examined the damaged zone in front of the tip of a chevron notched crack with SEM directly without preparation of thin sections. Thin-section specimens were not used in this study to avoid creating extra microcracks during preparation of the specimens.

Scanning electron micrographs of fracture surfaces of Brisbane tuff CCNBD specimens tested under cyclic loading are shown in Figure 7.42. The petrographic details of Brisbane tuff are given in Chapter 3. According to the petrographic analysis results, quartz and feldspar minerals are infrequently packed in silica cryptocrystalline cement. As shown in Figure 7.42, excessive dust and small particles are observed close to the tip of the chevron notch crack. Dust and small particles are not observed in regions far from the tip of the chevron notch crack. It is believed that this region without dust and small particles is the starting point of unstable crack propagation.



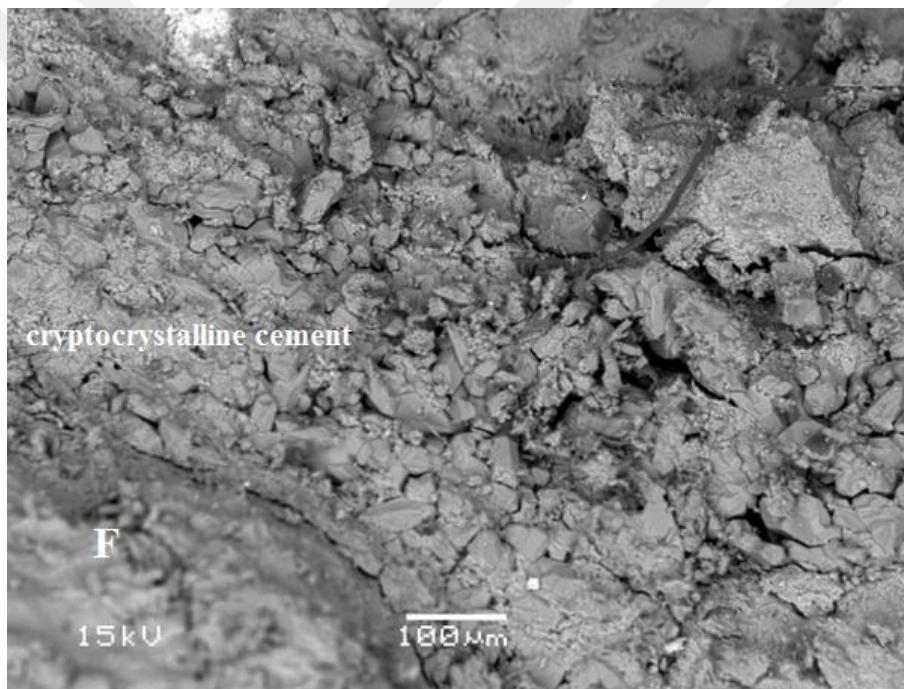
(a)



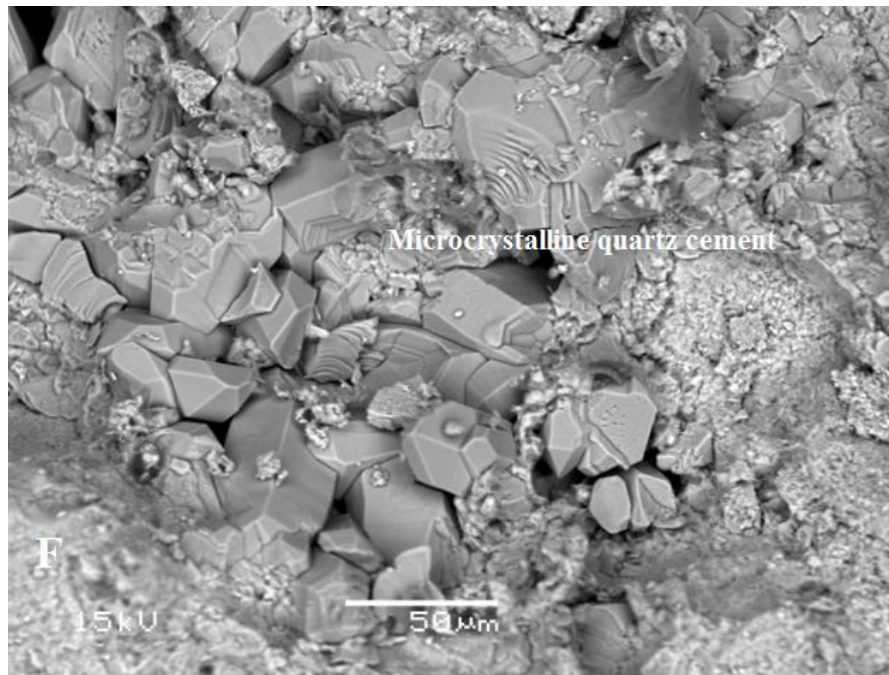
(b)

Figure 7.42 Debris and small particles in front of the tip of the chevron notch as a result of cyclic loading

In general, two fatigue mechanisms were observed in the cement of Brisbane tuff: (1) grain decohesion in secondary microcrystalline quartz cement intrusions (Figure 7.43) and (2) fatigue striations in primary silica cryptocrystalline cement (Figure 7.44). Figure 7.43 shows a closer look at the cement and feldspar mineral. Some microcrystalline quartz cement intrusions were seen in the primary silica cryptocrystalline cement after petrographic analyses. Figure 7.43b shows those microcrystalline quartz cement intrusions. Closer examination of the damaged cement would appear to indicate that a massive amount of loosened microcrystalline quartz minerals resulted from grain decohesion. Moreover, there is a clear grain boundary crack between feldspar and cement (Figure 7.43).



(a)



(b)

Figure 7.43 Fatigue damage in cement of Brisbane tuff resulting from decohesion of microscale quartz minerals

Because the CCNBD test is a diametral compressive test to induce indirect tensile stress, increasing the magnification of images reveals typical fatigue striations along the direction of maximum principal stress (compressive stress) in the area of the notch crack tip in cyclically tested CCNBD specimens (Figure 7.44).

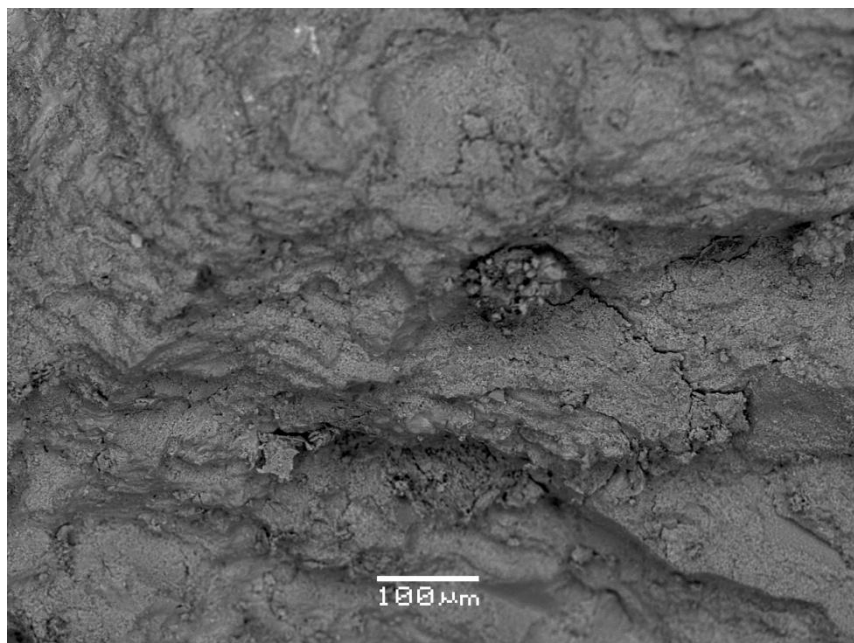
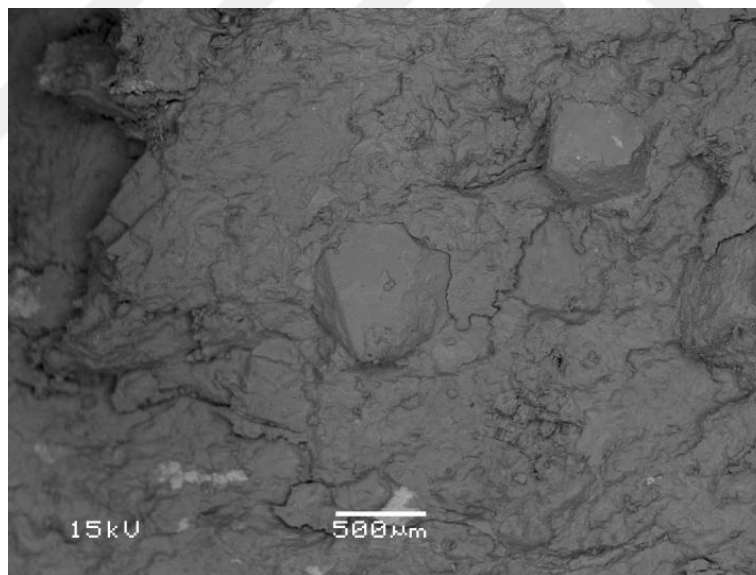
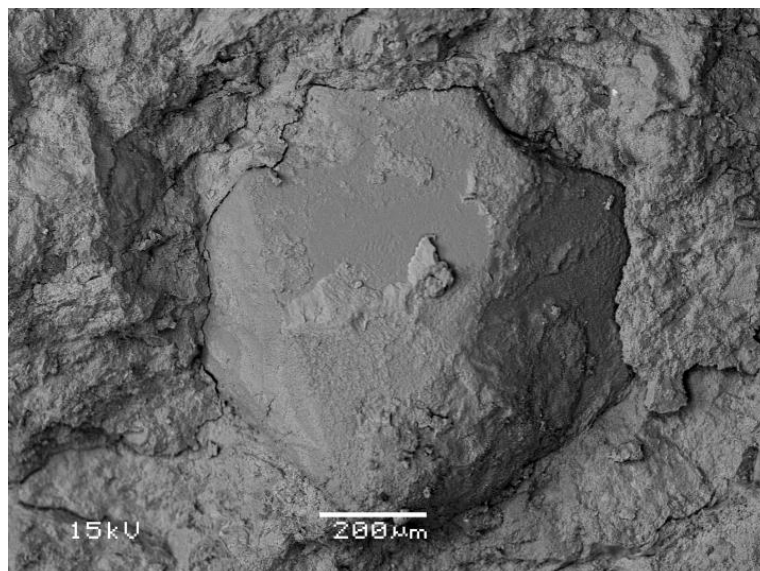


Figure 7.44 Fatigue striations in Brisbane tuff cement

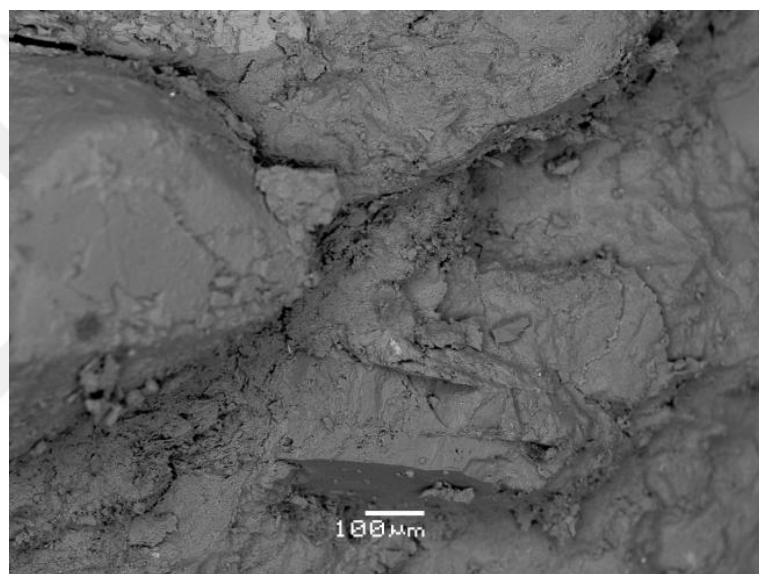
Detailed SEM imaging was done on Brisbane tuff rock forming minerals in addition to studying the fatigue damage mechanism in the cement. Similar to the fatigue damage in the cement, there were two fatigue damage mechanisms seen with grain-related damage: (1) intergranular cracks causing grain decohesion (primary mechanism) and (2) intragranular cracks (secondary mechanism). Brisbane tuff is composed mainly of quartz and K-feldspar minerals with small amounts of siderite (Fe-carbonate) and zeolite minerals. Figure 7.45 shows the fatigue-induced cracking around quartz and feldspar minerals. In general, fatigue-related microcracking around tuff minerals causing grain decohesion as a result of pulling out of grains by frictional sliding is clearly visible in Figure 7.45. A closer look ($\times 1000$ magnification) at the cement after pulling out a grain reveals scratches on the surface that may indicate that frictional sliding (Figure 7.45e). Figures 7.45b and 7.45c are presented at greater magnification in Figure 7.45e. At this scale intergranular cracking around grains is apparent. In greater detail, fine fragments around grains were observed, which accumulate in the proximity of the grain corners. This is particularly evidenced in Figure 7.45c, from which we can infer that such powder is the result of fatigue damage, probably due to local abrasion of the sharp corners of the quartz and feldspar grains.



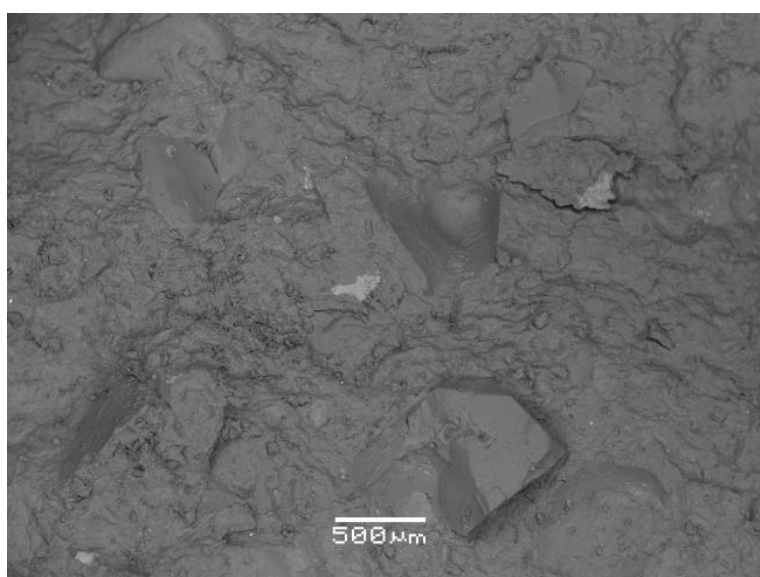
(a)



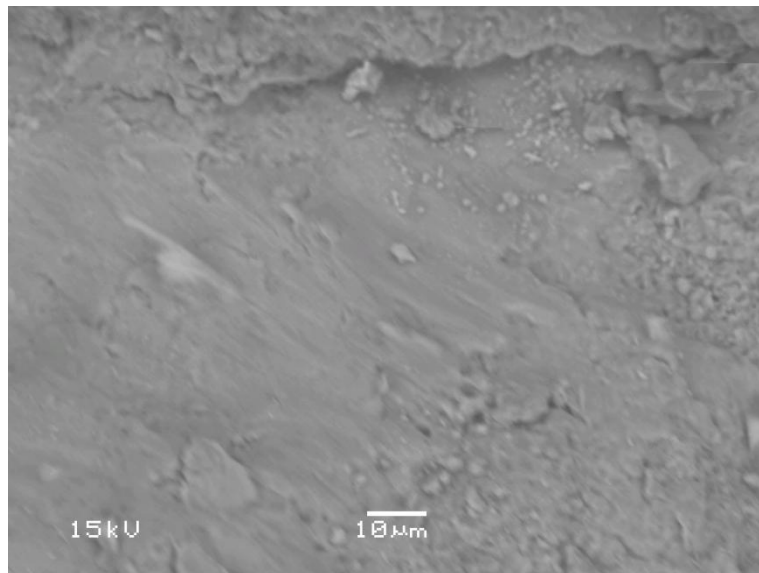
(b)



(c)



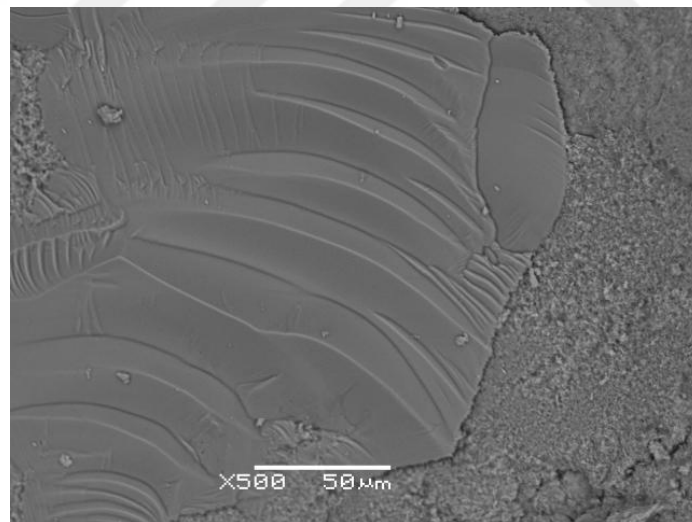
(d)



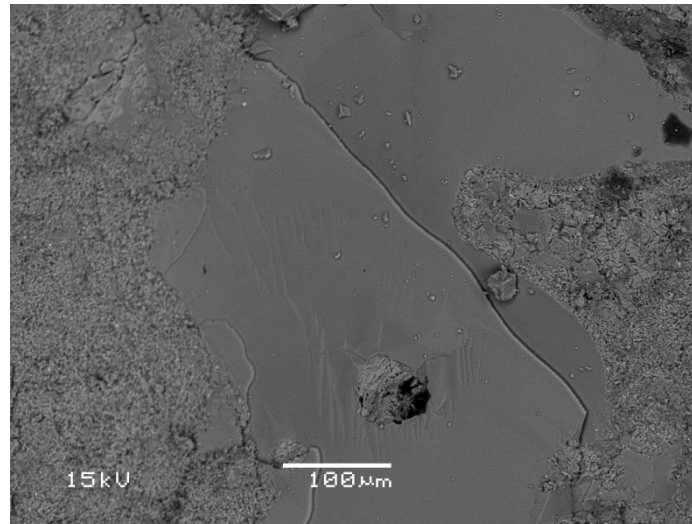
(e)

Figure 7.45 Grain decohesion and loosened grains (a), (b), (c), (d) and frictional scratches with high magnification ($\times 1000$) at the surface of fatigue crack (e)

Besides the intergranular cracks around grains, a much smaller amount of intragranular cracks were observed with SEM imaging in front of the chevron tip tested under cyclic loading. Figure 7.46 shows a cleaved quartz mineral with fatigue striations, and a transgranular crack.



(a)



(b)

Figure 7.46 Fatigue striations in damaged grains (a) and transgranular crack (b)

SEM imaging revealed that the primary fatigue damage mechanisms in front of a chevron notch crack are grain decohesion and intergranular cracks. Debris and dust are the results of fatigue damage in Brisbane tuff cement and around loosened grains. The fatigue cracks in the cement are restricted around the grains and cannot grow through the grains (Figure 7.47). It is perhaps true to say that those fatigue cracks are stable (subcritical) cracks that coalesce to form macroscale fatigue cracks resulting in failure. It is also possible to say that each grain after decohesion may behave as an indenter to indent the surface of the weaker cement material under far field cyclic diametral compressive loading. This could be an explanation for the mechanism of the debris material around and in the corners of loosened grains.

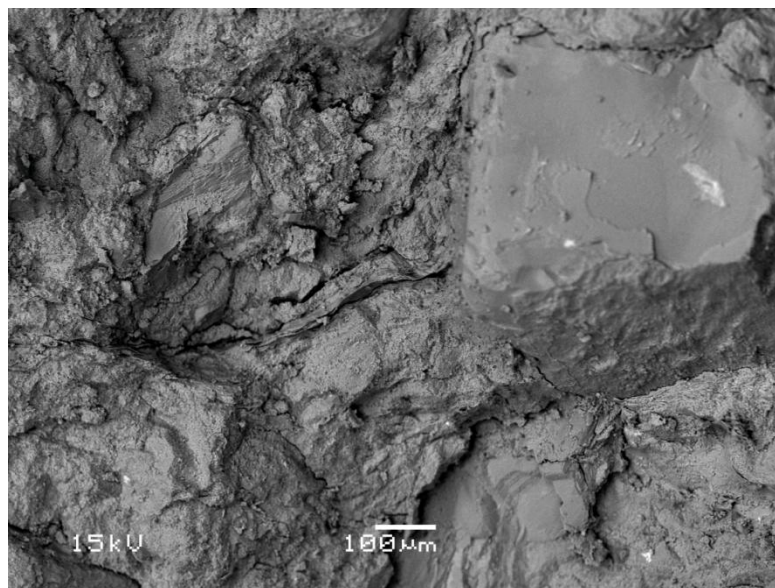
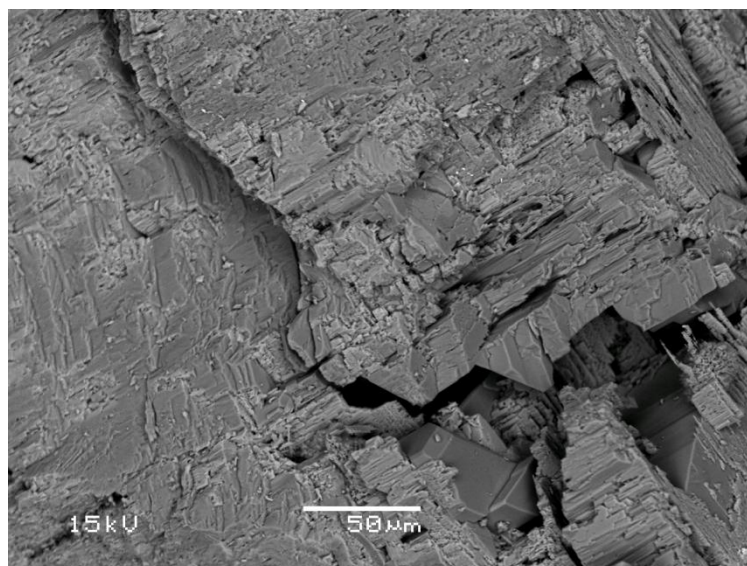


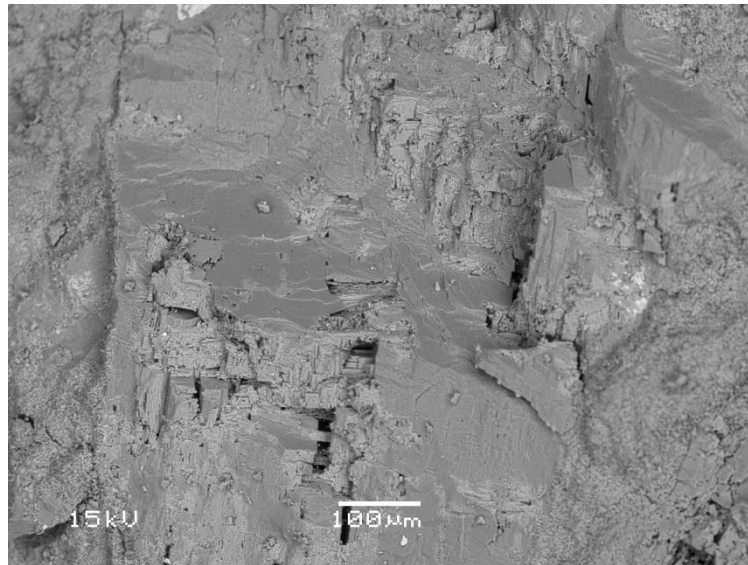
Figure 7.47 Cracks around mineral

Observations of fracture surfaces in front of the chevron crack tip with SEM indicated that the origin of such permanent strain seen by CMOD under cyclic loading can arise from a variety of inelastic deformation mechanisms, such as dislocation plasticity, microcracking, frictional sliding, or subcritical crack mechanism (creep). It is possible to say that stress corrosion cracking due to grain decohesion by the fatigue crack mechanism may induce fatigue damage in front of pre-existing Griffith cracks. Another possible mechanism is particle breakage under cyclic loading, which probably starts at contacts between particles and is accompanied by the production of very small fragments, probably due to frictional sliding within the weak matrix. This stage can be correlated with a steady progression of damage and produces a general ‘loosening’ of the rock, which is a precursor to the formation of intergranular cracks.

To investigate possible differences between static and fatigue failure, the fracture surfaces of monotonically tested CCNBD specimens were also observed by SEM. Scanning electron micrographs of fracture surfaces of Brisbane tuff CCNBD specimens tested under monotonic loading are shown in Figure 7.48. When compared with static rupture, the main differences are twofold: (1) the number of fragments produced is much greater under cyclic loading than under static loading, and (2) intergranular cracks are formed due to particle breakage under cyclic loading compared with smooth and bright cracks along cleavage planes under static loading (Figure 7.48). Obviously, the macroscale main crack causing failure is seen in cement without any dust or debris material. Typical sparkling cleavage cracks resulted when rupture of the crystals occurred along cleavage planes.



(a)



(b)

Figure 7.48 Damage in cement and cleavage cracks under monotonic loading

In comparing the grain cracking between cyclically and monotonically broken specimens, the main differences are the transgranular cracks that are the dominant damage mechanism in monotonically broken samples. Moreover, grain decohesion and pulled out grains were not observed in front of the chevron notch crack tip in monotonically broken CCNBD specimens (Figure 7.49).

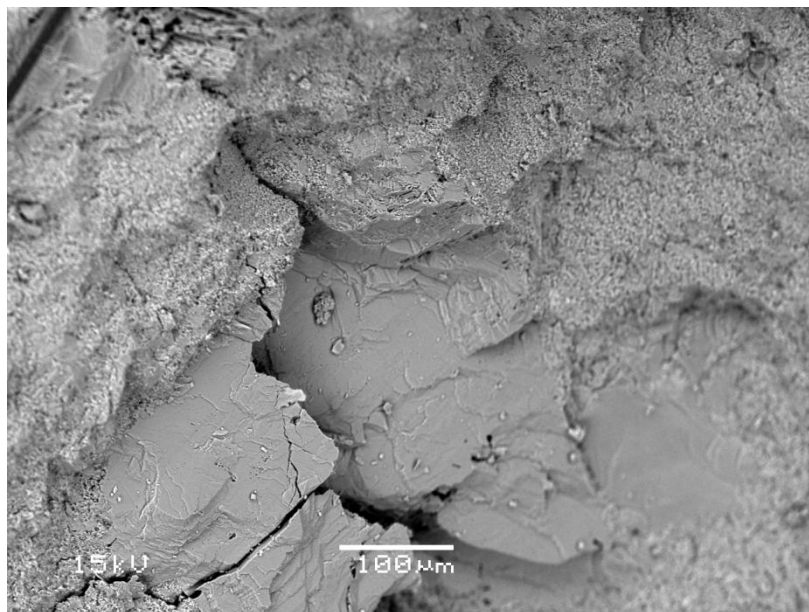


Figure 7.49 Damaged grains by transgranular cracking under monotonic loading

Compared with the load–CMOD plots of monotonically and cyclically tested CCNBD specimens shown in Figure 7.50, very interesting damage behaviour was observed with inclined chevron notch cracks under cyclic loading. The 45° and 70° chevron notch cracks start to open from the beginning

of the cyclic loading test, whereas in monotonic loading, chevron notch cracks start to close up to failure with all three test repeats. This outcome is a very important finding for examining rock fatigue damage mechanisms. In Section 7.5, it was shown that extra fatigue cracks initiated and propagated from the boundaries of the chevron notched crack in addition to the wing cracks around the tip of the notch crack.

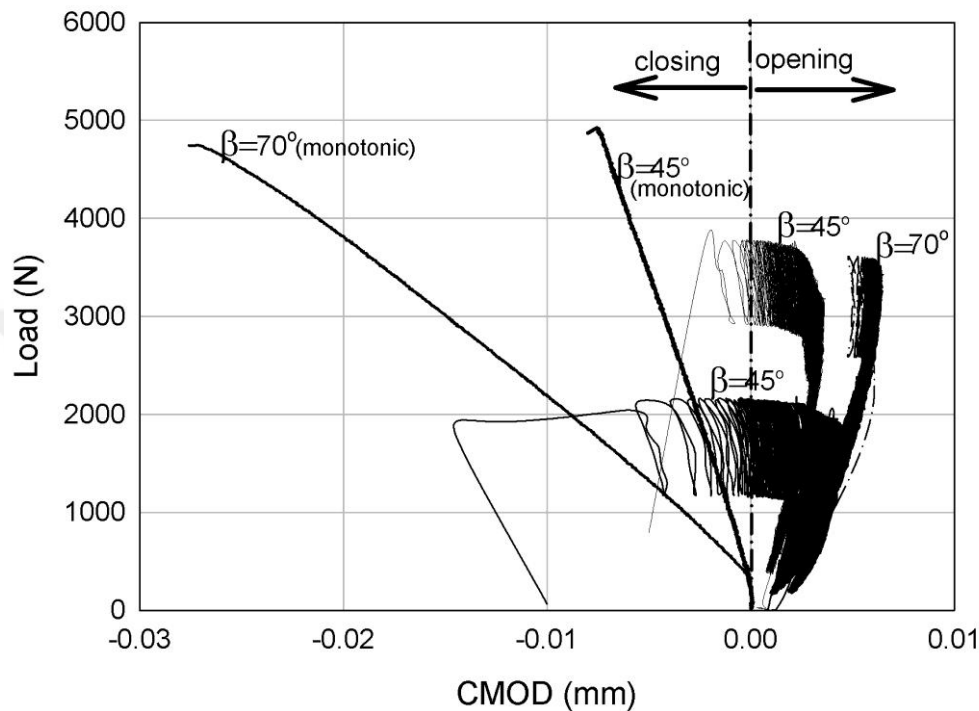


Figure 7.50 Comparison of notch crack deformation under monotonic and cyclic loading

On the basis of the fracture patterns in tested specimens and the SEM imaging, it is difficult to determine whether wing cracks start before fatigue cracks or vice versa. However, this outcome about notch crack opening under cyclic loading versus notch crack closing under monotonic loading could be explained by some approaches. Because crack opening starts at the beginning of cyclic loading, as shown in Figure 7.50, both wing cracks and fatigue cracks may initiate at the same time around the boundary of the notch crack during cyclic loading, causing the notch crack to open. After both wing and fatigue crack initiation, a possible wedge formation in front of the notch crack tip takes place to apply diametral compressive loading to open the chevron notch crack (Figure 7.51).

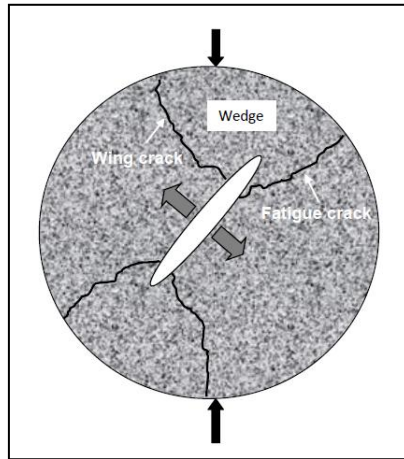


Figure 7.51 Wedge formation in front of the tip of notch crack after wing and fatigue crack initiation

Detailed SEM examinations were done on the surfaces of both fatigue and wing cracks on the CCNBD specimens tested with $\beta = 45^\circ$ inclined chevron notch crack under cyclic loading. Figure 7.52 shows an SEM image of fatigue cracking. It can be seen that grains are forced out of the cryptocrystalline matrix as the fatigue crack propagates, whereas some of them stayed in the cement because of grain boundary cracks. In general, it was observed that the fracture surfaces of fatigue cracks at the boundaries of inclined notch cracks are much rougher compared with surfaces of mode I fatigue cracks (Figure 7.52 and 7.53). Moreover, the damage to cement at the surface of fatigue cracks at the boundaries of inclined notch cracks is much worse compared with that to the cement at the surface of mode I fatigue cracks.

Figure 7.54a shows a grain damaged by a transgranular crack, and high magnification of the damaged cement around that damaged grain. It is again clear that grain decohesion is the dominant damage mechanism in cryptocrystalline cement.

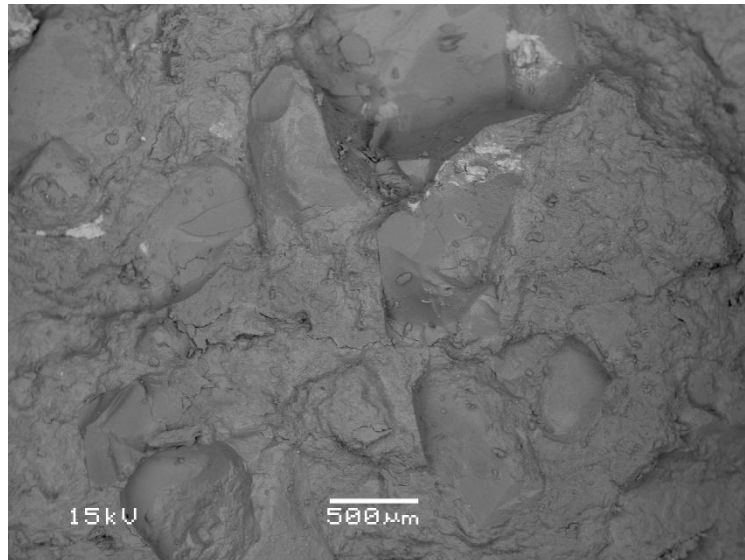


Figure 7.52 SEM image of fatigue crack near the tip of $\beta = 45^\circ$ inclined chevron notch crack

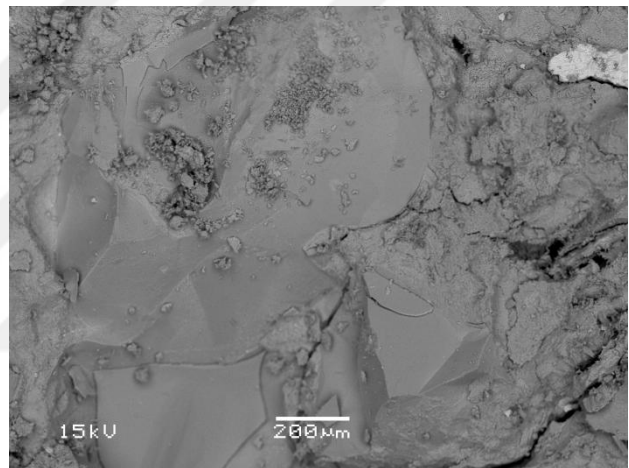
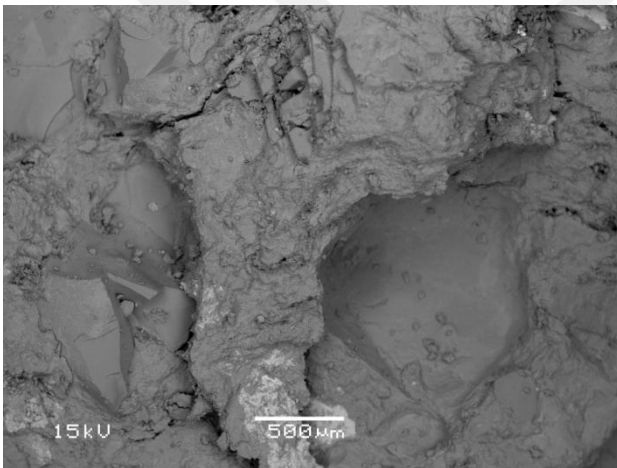


Figure 7.53 Damage to cement at the surface of fatigue crack

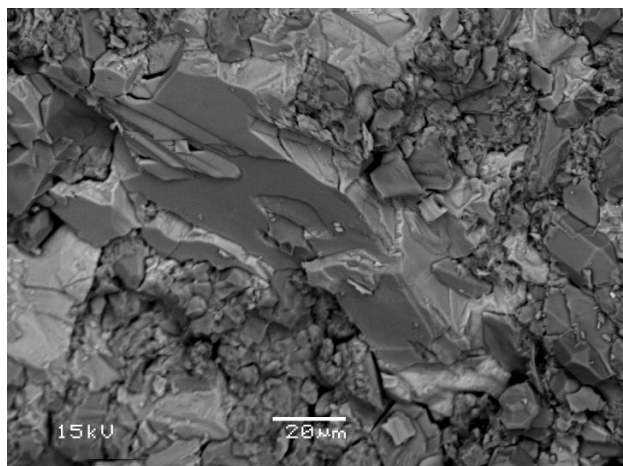
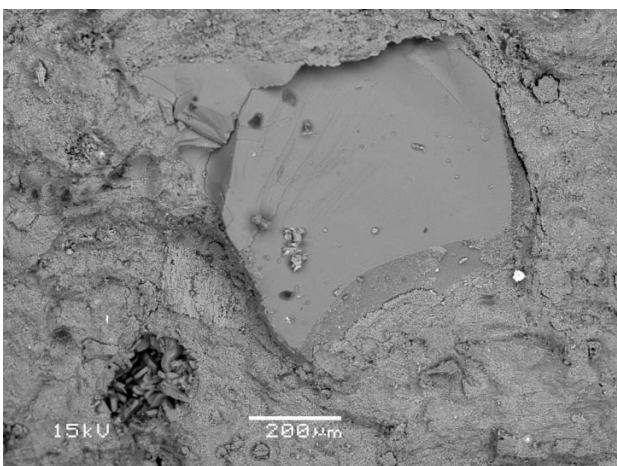


Figure 7.54 Damaged and loosened grain as a result of transgranular cracking (a, left) and damage to cement because of microscale quartz decohesion (b, right)

After SEM analysis, a question may arise: What do forced out or loosened grains do along fatigue crack surfaces during each loading and unloading cycle? It is possible to say that the rough fracture faces are pressed against one another by the shear displacement around the inclined notch cracks. Then, each grain can act as an indenter on weaker cement material to form typical indentation cracks during loading and unloading. Significant general plasticity or grain boundary sliding can then generate larger grain boundary cracks or cause a pre-existing crack to grow in a stable manner. A similar process for nucleation of cracks was found to be controlled by dislocation of grains in creep mechanism (Gandhi & Ashby, 1979).

7.5 Chapter Summary

The fracture toughness response to cyclic loading was found to be different from that under static loading in terms of the ultimate load and induced plastic displacement. The maximum reduction of the static K_{IC} of 46% was obtained for the highest amplitude dynamic cyclic loading tested. For sinusoidal cyclic loading, a maximum reduction of the static K_{IC} of 27% was obtained. These reductions clearly illustrate the dramatic effect of cyclic loading on the fracture resistance of cracks in rocks.

Damage was quantified as the accumulation of permanent strain in front of the chevron notch crack tip with each cycle of loading, because microfracturing introduces nonlinearity into the theoretically elastic behaviour of the rock. A continuous irreversible accumulation of damage was observed in dynamic cyclic tests carried out at different amplitudes. After the accumulation of irreversible damage and failure of the specimen, clear tensile softening was observed in cyclic loading tests carried out at different amplitudes on vertically aligned chevron notch cracks (mode I). However, no postpeak behaviour was observed in the CCNBD specimens tested under static loading. Considering the shape of load–CMOD plots, the accumulation of irreversible damage in front of the chevron notch crack under cyclic loading was found to be similar to static fatigue (creep). Moreover, the subcritical crack mechanism was verified from rock fatigue research, with lower ultimate loads causing smaller stress intensity values (K_I) than the critical stress intensity values (K_{IC}) due to rock fatigue.

In general, two different postpeak behaviours were observed, depending on the inclination of the notch crack in terms of tensile stress softening after the postpeak region in load–CMOD plots. Very clear tensile softening was observed with vertically aligned notch cracks (mode I), whereas no tensile softening was observed with inclined notch cracks (mixed-mode I–II). However, a common

outcome can be seen from load–CMOD plots. The change in notch crack stiffness during loading–unloading cycles can be separated into two distinct regimes: (1) irreversible damage accumulates constantly up to the plastic part of the load–CMOD curves, suggesting that formation of subcritical cracks (stable cracks) occur and (2) the stiffness of the notch crack decreases as considerable microcracking forms an extensive process zone in front of the notch crack tip, suggesting that high acceleration of the growth rate of subcritical cracks takes place.

The SEM results enable some of the qualitative features of the fatigue damage process in Brisbane tuff to be inferred. It was clearly understood after SEM analysis that rock fracturing depends strongly on its microstructures and loading situation. Because of the delicate balance between the stress required to cause a mineral grain to cleave and the stress required to cause brittle grain boundary cracking it is not always an easy matter to predict which will predominate under any given set of conditions. This balance can be upset by small changes in loading type, degree of heterogeneity, cement mineralogy and rock texture. In general, it was found from this research that the failure of a CCNBD specimen under cyclic loading is a result of the coalescence of many microcracks, not of the growth of a single macrocrack. SEM images showed that fatigue damage in Brisbane tuff is strongly influenced by the failure of the matrix because of both intergranular fracturing and transgranular fracturing. The main characteristic is particle breakage under cyclic loading, which probably starts at contacts between particles and is accompanied by the production of very small fragments, probably resulting from frictional sliding within the weak matrix. It is believed that point contacts at grain boundaries are regions of stress concentration (i.e., indenters). Transgranular cracks may emanate from these regions, and intergranular cracks sometimes pass through the contact points. This stage can be correlated with a steady progression of damage and produces a general ‘loosening’ of the rock, which is a precursor to the formation of intergranular cracks.

CHAPTER 8

DISCUSSION

Rock mechanics generally is concerned with failure in a continuum sense, where rock or a rock mass undergoes permanent damage, thus affecting its ability to sustain a load. Fracture mechanics, sometimes called crack mechanics, is a tool that relates understanding of how fractures (cracks) originate, propagate, and terminate in a given material under given stress. An understanding of the fracture mechanism for rocks is essential for mining machinery design, mining excavation, tunnelling, blasting and civil engineering structures. The most fundamental parameters in fracture mechanics are fracture toughness, which describes the resistance of a material to crack propagation, and tensile strength of a material. Hence, the research presented in this thesis was undertaken to investigate in detail the indirect tensile strength and fracture toughness properties of Brisbane tuff relevant to monotonic and cyclic loading.

In both loading types, Brazilian disc specimens were used. However, the Brazilian test with or without a pre-existing crack has been criticised ever since it was initially proposed. Although the Brazilian test has been studied extensively, both experimentally and theoretically, relatively little attention has been directed towards researching the validity of the test. For example, in this research, the standard Brazilian indirect tensile test caused catastrophic crushing failure of the disc specimens, rather than the expected tensile splitting failure initiated by a central crack. This finding led to an investigation of the effect of loading conditions on the failure of Brazilian disc specimens by using three steel loading arcs of different angles applied to three different rock types.

So far, rock fatigue research has been done using cylindrical specimens to investigate the degradation of uniaxial compressive strength of rocks due to fatigue. However, the failure mechanism of cylindrical rock specimens is due to tensile cracks. Thus, the effect of rock fatigue on indirect tensile strength and fracture toughness was investigated in this research to find the damage mechanism of rock fatigue.

8.1 Indirect Tensile Rock Fracturing under Static Loading

A key objective of this study was to analyse the tensile stress distribution and failure of disc specimens under loading arcs of different angles, in comparison with results of the standard Brazilian test. All experimental, numerical and analytical results obtained in this research show that

the formula for calculating indirect tensile strength from the failure load obtained by the standard Brazilian test might not represent the actual tensile strength of brittle rocks. This is because the contact area between the standard Brazilian loading jaws and the rock disc specimen cannot be considered as a line.

In the experiments described herein, the disc specimens shattered into several pieces under the standard Brazilian jaws. Moreover, the occurrence of several fractures branching from the diametral plane gives rise to doubt about the mechanism of failure in this test. It has been suggested that failure starts by shear fracture in the region of high compressive stresses near the contacts (Jaeger & Cook, 1976). In contrast, applying distributed diametral loads by using loading arcs causes single failure planes along the loaded diameter in all three rock types tested. According to Jaeger and Cook (1976), shear fracture at the contacts can be decreased if the load is applied to the circumference of the cylinder. They stated that experiments with distributed loads over a narrow arc, usually about 15° , yield values for the tensile strength that are little different from those obtained with line loads and give rise to similar diametral extension fractures (Jaeger & Cook, 1976).

According to the Griffith criterion (1920) used for failure in solid discs, the exact centre of the disc is the only point at which the conditions for tensile failure at a value equal to the uniaxial strength are met. This result is also indicated by analytical solutions given by Hondros (1959). Fairhurst (1964) generalised the Griffith criterion to account for variation of the compression/tension strength ratio (n) from the theoretical value, $n = 8$. Fairhurst's (1964) results indicate that in order to assure fracture initiation near the centre of a homogeneous specimen it is necessary to spread the applied load over an appreciable arc of contact (20° or more). He suggests that with a narrow contact strip ($\alpha = 5^\circ$) there will be a pronounced tendency for off-centre fracture initiation in rocks that have a low compression/tension strength ratio. He also showed that if compression/tension strength ratio is relatively small and the contact strip is narrow, there will be a systematic underestimation of tensile strength if fracture does occur near the centre of the specimen. Our experimental and numerical results are in good agreement with Fairhurst's (1964) results.

The indirect tensile strength of Brisbane tuff was calculated as 9.2 MPa using the formula proposed by ISRM (Table 8.1). According to the analytical solutions of Hondros (1959), when $2\alpha < 11^\circ$ and r/R tends to zero, the tangential stress along the vertical diameter may be calculated using the same formula. This indicates that the formula proposed by ISRM is not just for line loading but also for diametral loading over an arc of up to $2\alpha = 11^\circ$. This outcome was also verified by Jaeger and Cook (1976). They found from experiments with distributed loads over a narrow arc, usually up to about

$2\alpha = 15^\circ$, that the tensile strength varied little from that obtained using a line load and gave similar diametral extension fractures. In contrast, the tensile strength decreases to 6.61 MPa when $2\alpha = 15^\circ$ is used in theoretical solutions (Table 8.1). This result raises another doubt about the applicability of the formula proposed by ISRM to determine the indirect tensile strength of rocks.

Table 8.1 Comparison of experimental, numerical and analytical results for Brisbane tuff

Loading mode	Experimental ultimate load (kN)	Projected width loaded section of disc (mm)	Theoretical of tensile stress at centre of disc (MPa)	Numerical tensile stress at centre of disc (MPa) (FRANC2D)	Numerical tensile stress at centre of disc (MPa) (RFPA2D)
Standard	19.60	1.00	9.20	12.36*	7.91
Brazilian jaws					
15° loading arc	14.81	6.80	6.61	6.88	8.04
20° loading arc	19.20	9.07	8.21	8.79	8.29
30° loading arc	22.30	13.60	8.85	8.98	8.70

* Calculated assuming $\alpha = 1^\circ$

Very few studies have been done to investigate contact loads in terms of edge effects between loading arcs and specimens (Wang *et al.*, 2004; Yanagidani *et al.*, 1978). In general, it is assumed that load is applied by loading arcs radially and uniformly on the sample in both closed-form analytical solutions and numerical modelling in literature. However, load is not uniform and radial over all contact points between the arc and the specimen, hence, it is necessary to investigate the contact stresses in terms of edge effects of the corners of the steel arcs and frictional forces between the sample and the loading arcs under different geometries.

8.2 Fracturing Modes and Influencing Factors

Diametral compressive load is a kind of compressive-shear loading across a crack plane. However, no shear crack parallel to the crack plane was observed experimentally in all three rock types used in this research up to 70° crack inclination angle except for 70° inclined sandstone and granite samples. In contrast, the highest failure load was seen with the 33° crack inclination angle for all three rock types. In this situation the question arises of whether the 33° crack inclination angle is optimal for possible shear cracking parallel to the crack plane. However, pure shear loading was not

found to result in true mode II fractures in some studies. For example, Huang and Wang, (1985) and Swartz and Taha (1990) found that the central crack initiated at the notch tip propagates by deviating from the original notch plane at an angle of 50°–70° and the fracture was of mode I in their common mode II fracture toughness (K_{IIC}) tests.

FRANC2D was successful at simulating the crack initiation and propagation path with CCNBD geometry under diametral compressive loading. The simulation and experimental results are in good agreement in terms of crack initiation and crack inclination angle comparisons. However, mode I stress intensity factor values were found to be always higher than mode II stress intensity factor values. According to the mixed-mode fracture criteria, the mode I maximum stress intensity factor reaches its critical value (K_{IC}) when the mode II stress intensity factor equals zero ($K_{I \max} = K_{IC}$, $K_{II} = 0$) for pure tensile loading. In contrast, the mode II maximum stress intensity factor reaches its critical value (K_{IIC}) when the mode I stress intensity factor equals zero ($K_{II \max} = K_{IIC}$, $K_I = 0$) for pure shear loading. However, one of the most important mixed-mode criteria, maximum tangential stress criteria, was derived for a crack under monotonic tension. Hence, the predicted K_{IIC} values from the maximum tangential stress criterion are smaller than K_{IC} . However, for brittle rocks, shear resistance has been accepted to be higher than the tensile strength of rocks, thus, K_{IIC} should be larger than K_{IC} of rocks. In some research, it has been accepted that the mixed-mode fracture criteria are incapable of predicting mode II fracture and for the standard methods of measuring mode II fracture toughness (Backer, 2004; Rao *et al.*, 2003).

Wing cracks are tensile and appear under the action of tensile stresses caused by mutual gliding of opposite faces of the inclined chevron notched crack. The original crack (prior to the branching of the secondary cracks or wings) is a transverse shear (mode II) crack. The continuation of this gliding makes further stable growth of these branching cracks possible when loading proceeds. There is virtually no constraint for such gliding under resolved shear produced by axial compression (Dyskin, 2004). The simplest model for such a crack was proposed by Fairhurst and Cook (1966). Fairhurst and Cook (1966) used the inclined crack of the wing crack model as a mechanism to generate local tensile forces (F) normal to the direction of compressive loading. These forces originate from sliding of the opposite sides of the contact area. It is supposed that in brittle rocks a crack grows as a result of concentration of tensile stresses at its tips, i.e., as a mode I crack. Hence, only horizontal projections of these forces F tending to open the crack are taken into account. It should be noted that it is the contact area that produces these wedging forces, F . Without the contact, force F would be equal to zero. However, an initial crack with contacting faces is only

an extreme case of a defect capable of transferring the applied compression into local tension (Germanovich *et al.*, 1994).

8.3 Indirect Tensile Strength Degradation under Cyclic Loading

Sinusoidal and two types of increasing cyclic loading tests were conducted on Brazilian disc specimens. The reduction of ITS was found as 30% with sinusoidal loading tests, whereas the other two types of increasing cyclic loading cause reduction in ITS of a maximum 35% compared with ITS values under monotonic loading. Besides the decrease in ITS value, the fracturing mechanism of rock material was found to be very different under monotonic and cyclic loading because excessive amounts of dust and small particles were found with cyclically failed specimens. This observation suggests that the effective fracturing mode is not just mode I (opening) tensile fracturing in rock fatigue damage, although diametral compressive loading on a disc is suitable for producing indirect tensile stresses along a failure plane in the disc specimen. Because all pre-existing cracks and deficiencies in the rock disc specimen do not align perfectly parallel to the principal stress direction, shear type (mode II) or mixed type (mode I–II) fracturing mechanisms are possible around inclined pre-existing cracks in a disc specimen. Hence, sliding/shear effects of crack surfaces are believed to be the source of excessive microfracturing in cyclic tests.

SEM results showed that transgranular and intergranular cracks are the main mechanism of rock fatigue. It was reported in fatigue research that a massive amount of microfractures and irreversible damage accumulation are the main causes of rock fatigue (Carfagni & Salvatore, 1998; Holcomb, 1979; Pino *et al.*, 1989). SEM images showed many more small particles on failure surfaces in this research, too. In addition, transgranular and intergranular cracks are found to be the main damage mechanism of fatigue in our research. However, it is still unknown how the load cycles break the bonds between grains and rock cement to produce intergranular cracks around grain boundaries. SEM images showed that there are small particles and dust around grain boundaries. Two possible mechanisms can be drawn from SEM image analysis in this research: (1) Stiffer grains (quartz and feldspar minerals) act on the surface of the weaker cement as indenters under principal compressive stress. (2) There is an effective sliding/shear mechanism between grains and rock cement during each loading and unloading cycle where internal stresses are produced between grains and cement as a reaction to the external diametral compressive stress.

8.4 Fracture Toughness Degradation under Cyclic Loading

Fracture toughness (K_{IC}) is an important material property that corresponds to the critical state of the stress intensity factor required for crack initiation and subsequent propagation. Two different types of cyclic loading were used in this research; cyclic loading with constant mean level and constant amplitude, termed sinusoidal cyclic loading; and cyclic loading with increasing mean level and constant amplitude, termed increasing cyclic loading. The cyclic loading tests were carried out on CCNBD rock specimens. Crack displacement was measured in terms of the CMOD. The fracture toughness response to cyclic loading was found to be different from that under static loading in terms of the ultimate load and damage mechanism in front of the chevron crack. A maximum reduction of the static K_{IC} of 46% was obtained for the highest amplitude dynamic cyclic loading tested. For sinusoidal cyclic loading, a maximum reduction of the static K_{IC} of 27% was obtained. These reductions clearly illustrate the dramatic effect of cyclic loading on the fracture resistance of cracks in rocks.

Tensile stress in front of a crack tip is used to calculate fracture toughness of a material. Thus, parallel outcomes between the reduction in tensile strength of rock under cyclic loading and reduction of fracture toughness of the rock was expected. However, the percentage of reduction of each test was found to be different, 35% reduction of ITS and 46% reduction of fracture toughness due to rock fatigue. The reason for this difference comes from the geometry of the Brazilian disc specimen. The Brazilian disc specimen used for fracture toughness tests has a macroscale chevron notched crack representing natural defects and pre-existing cracks in rocks. In contrast, the Brazilian disc specimen used for ITS tests has no pre-existing macroscale notched crack. Disc specimens with chevron notched crack geometry were used in this thesis to investigate the behaviour of the FPZ in front of the pre-existing crack tips under cyclic loading.

SEM images of chevron notch crack tips showed similar results with to those found in ITS test specimens. Transgranular and intergranular cracks are found to create extra microcracks for irreversible strain accumulation due to fatigue in specimens. However, irreversible strain accumulation under cyclic loading starts at a very early stage of loading compared with monotonic loading. This may mean that decohesion and loosening between grains and cement start with that early loading stage. This outcome can be supported by the behaviour of 45° inclined notched cracks under cyclic loading, as given in Chapter 7. Because this loosening starts at a very early stage of loading, crack faces of the notched cracks start opening under cyclic loading whereas crack faces of notched cracks start closing under monotonic loading.

CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

In this research, deformation and direct/indirect tensile strength characteristics of brittle rocks, mainly Brisbane tuff, were investigated. The results of cyclic loading tests for tensile fatigue limit or tensile fatigue strength found in this study are expected to be used in surface and underground design instead of the commonly employed indirect tensile strength values determined under monotonic loading. The basic methodology of this research involves mainly laboratory testing and numerical simulations. The mechanism of the tensile failure of rock under diametral compressive monotonic and cyclic loading was investigated in detail using Brazilian disc specimen tests and Chevron Notch fracture toughness tests.

Through the combined use of strain gauges, crack mouth opening transducers and SEM images, brittle rock damage mechanisms were investigated under different types of loading such as diametral compressive monotonic/cyclic loading, which produces indirect tensile stress in specimens. In addition, numerical tools FRANC2D and RFPA2D were used to investigate the tensile cracking mechanism, such as mode I (tensile) and/or mixed-mode I–II (tension-shear) fracturing modes, in rock under monotonic diametral compressive loading.

Once the lower failure load limit was obtained with cyclic loading tests compared with relative test types under monotonic loading, direct SEM imaging was performed on fracture surfaces of failed specimens. Results were used to identify the rock fatigue damage mechanism by making comparisons between the damage under monotonic loading and cyclic loading.

9.1 Conclusions

In this thesis, ISRM-suggested rock mechanical characterisation tests on Brazilian disc specimens with notch or without notch were performed under monotonic loading first, and then outcomes of these tests were used as input data for subsequent cyclic loading tests. The following conclusions are drawn from this thesis:

- From the indirect tensile tests on Brisbane tuff, granite and sandstone specimens, it was concluded that both tensile fracturing and indirect tensile strength of rocks are strongly dependent on the type of diametric loads applied. A general increase in failure load was

obtained with increasing angle of loading arc for all three rock types tested. The maximum recorded failure load, corresponding to the finest and most centrally located crack, was obtained with a $2\alpha = 30^\circ$ loading arc. The highest standard deviation for the failure loads was obtained using the standard Brazilian jaws, which resulted in catastrophic specimen failure. The direct tensile strength of Brisbane tuff was found to be lower than the indirect tensile strength, as a result of the difficulty in eliminating torsional effects in the direct tensile test. The highest failure load was found experimentally and through numerical modelling with two different programs, FRANC2D and RFPA2D, at a 30° loading arc.

- Fracture toughness research showed that CCNBD testing has numerous advantages over SR testing, including easier specimen preparation, a simpler testing procedure, a much higher failure load, and easy adaptation to mixed-mode fracture toughness testing. In addition, it is also unnecessary to perform pre-cracking for the CCNBD specimen, because it uses a chevron notch, which leads to stable crack propagation. Of the two mode I fracture toughness testing methods (SR and CCNBD), the CCNBD test has the smallest standard deviation in both its recorded maximum load and mode I fracture toughness. The mode I fracture toughness determined using the CCNBD test is smaller than that determined using the SR method. To verify the exact fracture toughness value of the tested rocks, the results of the SR and CCNBD tests were compared with some physico-mechanical rock mechanical parameters, including the uniaxial compressive and tensile strengths of rock specimens. The results showed that the SR method gives a higher fracture toughness compared with the CCNBD method.
- According to the experimental load–CMOD curves and numerical results, stable crack propagation is possible at the tip of the notch, prior to unstable crack propagation and failure. It was found that the critical crack length (a) is between the inner chevron notch crack length (a_0) and the outer chevron notch crack length (a_1) of the CCNBD specimen. That is, the crack front width increases linearly from zero to a point less than thickness (B) of the specimen. The analytical results and numerical simulation results were in agreement. Consequently, FRANC2D was found to be quite useful and practical for stress intensity factor analyses of the crack tip and for discrete crack propagation modelling of mode I fracture toughness.
- To investigate mixed-mode rock fracturing under monotonic loading, disk specimens of three rock types, Brisbane tuff, granite and sandstone, were diametrically loaded with various

angles (β) inclined with respect to the diametral compressive loading direction to investigate mixed-mode rock fracturing. The highest failure load was obtained for a 33° inclined notch crack in all three rock types. Also, typical wing cracks were obtained at the tip of all the notch cracks inclined at various angles from $\beta = 28^\circ$ to $\beta = 70^\circ$ in all three kinds of CCNBD rock specimens. However, no typical shear crack was obtained in a parallel direction to the notch crack plane. It was concluded that it may not be possible to get mode II (shear) fracturing with CCNBD specimen geometry under diametral compressive loading at any notch crack inclination angle. Moreover, cracks initiated at the tip of the notch for all three kinds of rock specimens with values of β up to 30° only. However, when the inclination angle is bigger than 33° , the crack initiation point moved from the tip to the centre of the notch crack with increasing inclination angles.

- Numerical modelling results obtained from mixed-mode I–II rock fracturing simulations show that both tensile and shear stresses are effective at the face and tips of the inclined chevron cracks for all crack inclination angles. The maximum tensile stress was obtained at the tip of the $\beta = 0^\circ$ inclined chevron notch crack. The obtained maximum tensile stress is 38 MPa higher than the indirect tensile strength of the rock. The highest shear stress was obtained with $\beta = 30^\circ$ and $\beta = 33^\circ$ inclined cracks, at 17 MPa. In contrast, the smallest shear stress was obtained with $\beta = 0^\circ$ inclined crack.
- From the S–N curve for Brisbane tuff, the indirect tensile strength was found to reduce from 10.8 MPa to 7.5 MPa as a result of cyclic loading. A second series of cyclic loading tests was performed on Brazilian disc specimens with increasing mean level cyclic loading. From results of the sinusoidal and increasing cyclic loading, increasing cyclic loading seems to be more effective in forcing ITS degradation of rocks than sinusoidal cyclic loading. Sinusoidal cyclic loading reduced the ITS by 30%, while dynamic cyclic loading reduced the ITS by 35%.
- The fracture toughness response of rock to cyclic loading was found to be different from that under static loading in terms of both the ultimate load and the induced plastic displacement. The maximum reduction in the static fracture toughness K_{IC} (of 46%) was obtained for the highest amplitude dynamic cyclic load applied. For sinusoidal cyclic loading, a maximum reduction from the static K_{IC} of 27% was obtained. These reductions clearly illustrate the dramatic effect of cyclic loading on the fracture resistance of cracks in rocks.

- Damage was quantified as the accumulation of permanent strain in front of the chevron notch crack tip with each cycle of loading, because microfracturing introduces nonlinearity into the theoretically elastic behaviour of the rock. A continuous irreversible accumulation of damage was observed in dynamic cyclic tests carried out at different amplitudes. After the accumulation of irreversible damage and failure of the specimen, clear tensile softening was observed in cyclic loading tests carried out at different amplitudes on vertically aligned chevron notch cracks (mode I). However, no postpeak behaviour was observed in the CCNBD specimens tested under static loading. Considering the shape of the load–CMOD plots, the accumulation of irreversible damage in front of the chevron notch crack tip in cyclic loading was found to be similar to static fatigue (creep). Moreover, the subcritical crack phenomenon was verified with lower stress intensity values (K_I) than the critical stress intensity values (K_{IC}) as a result of rock fatigue.
- The SEM results enable some of the qualitative features of the fatigue damage process in Brisbane tuff to be inferred. It was clearly understood after SEM analysis that the fracture of rock depends strongly on its microstructure and the loading conditions. Because of the delicate balance between the stress required to cause a mineral grain to cleave and the stress required to cause brittle grain boundary cracking, it is not always easy to predict which will predominate under any given set of conditions. This balance can be upset by small changes in loading type, degree of heterogeneity, cement mineralogy and rock texture. In general, it was found from this research that the failure of CCNBD specimens under cyclic loading is the result of the coalescence of many microcracks, not the growth of a single macrocrack. SEM images showed that fatigue damage in Brisbane tuff is strongly influenced by the failure of the matrix, because of dominant intergranular and transgranular fractures. The main characteristic is particle breakage under cyclic loading, which probably starts at contacts between particles and is accompanied by the production of very small fragments, probably due to frictional sliding within the weak matrix. It is believed that point contacts at grain boundaries are regions of stress concentration (i.e., indenters). Transgranular cracks may emanate from these regions, and intergranular cracks sometimes pass through the contact points. This stage can be correlated with a steady progression of damage and produces a general ‘loosening’ of the rock, which is a precursor to the formation of intergranular cracks.

9.2 Recommendations for Future Research

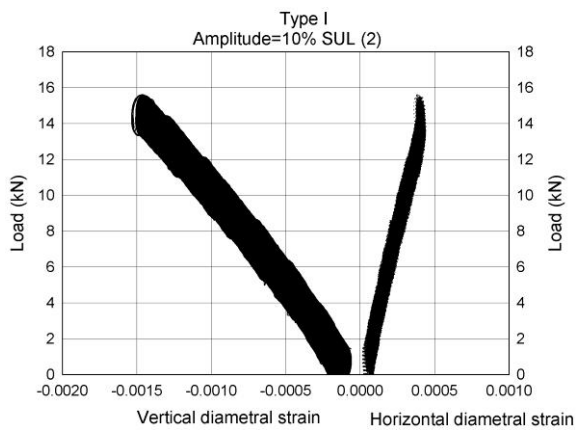
Extension of this work and continued studies relating the damage mechanisms of three mode types (mode I, mode II and mixed-mode I–II) rock fracturing and indirect tensile fracturing under monotonic and cyclic loading could be addressed as follows:

- Test results from this study revealed that indirect tensile strength of rocks increases with increasing loading arcs. Moreover, central tensile fracturing was found to be dependent on the types of diametral compressive loading. However, further testing could be performed to determine the effects of friction between loading arcs and disc specimens on the initiation and propagation of central tensile cracks in Brazilian disc specimens.
- The numerical modelling study performed in this thesis was done using two-dimensional numerical tools. Although a CCNBD disc specimen is thought to provide plane stress conditions, the geometry of CCNBD could be accepted as a three-dimensional problem because of the chevron tip inside the CCNBD specimen. Further three-dimensional numerical simulations are recommended to investigate mixed-mode I–II rock fracturing, especially.
- The cyclic tests performed in this study could be duplicated with different rock types. This would allow investigating the relationship between rock fatigue damage mechanism and grain–cement interaction with different grain sizes and different strength matrix relationships.
- The effect of increasing cyclic loading on tensile strength properties of rocks was investigated in this thesis with varying amplitude but at a constant frequency, 1 Hz, because of the limitations of the loading machine. Therefore, in future research, using varying frequencies is recommended for investigating the effect of frequency with increasing cyclic loading on rock fatigue.

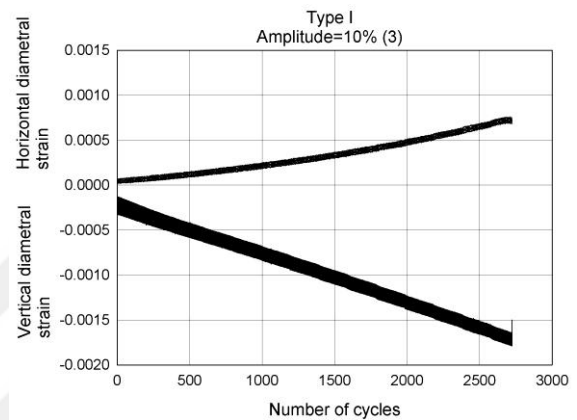
APPENDIX A

Degradation of Indirect Tensile Strength of Rocks through Cyclic Loading

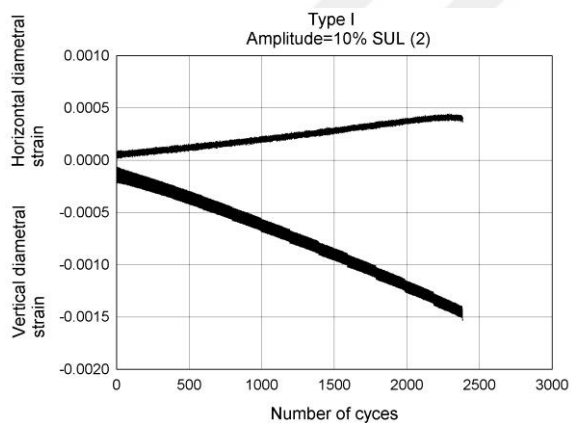
Effect of Type I cyclic loading on indirect tensile strength of Brisbane tuff



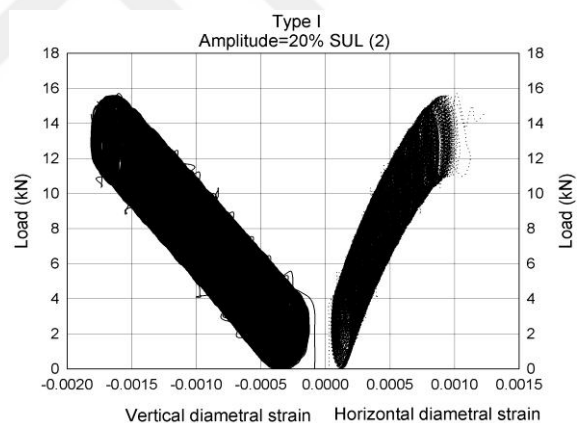
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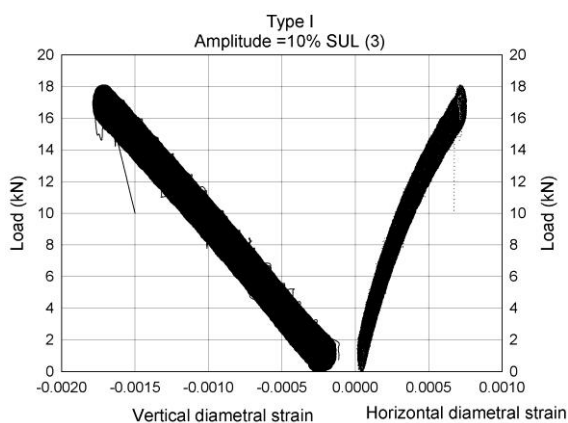
(d)



(b)

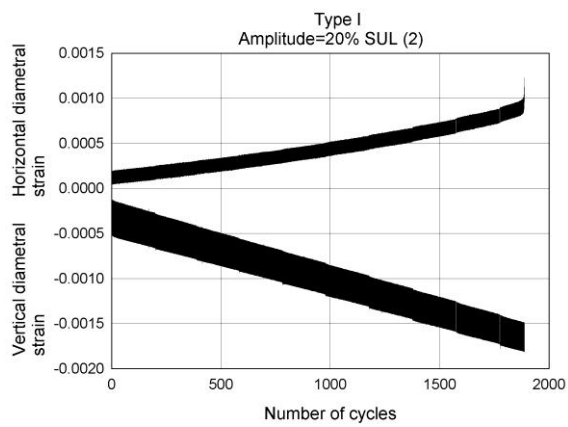


(e)

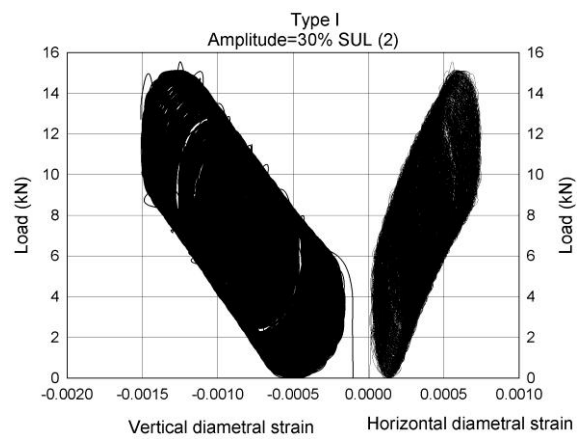


(c)

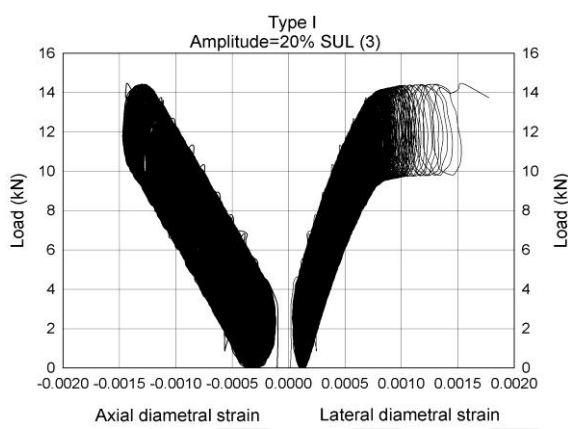
Figure A.1 Type I diametral cyclic loading; amplitude = 2.3 kN (10% SUL) (a, b, c, d and e)



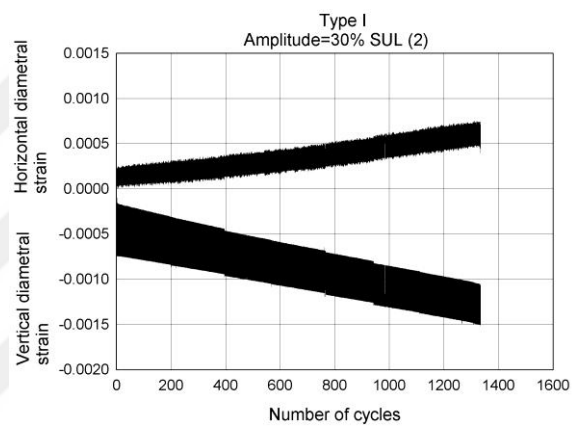
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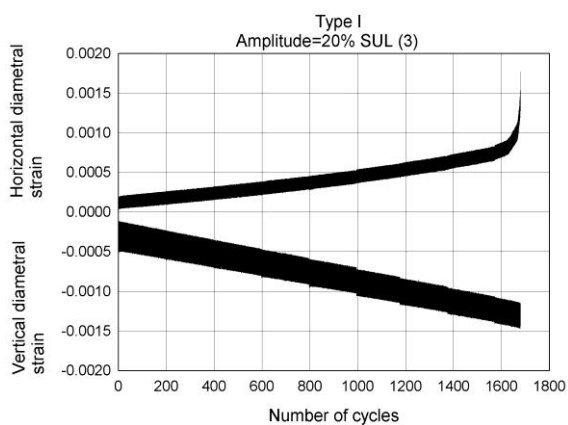
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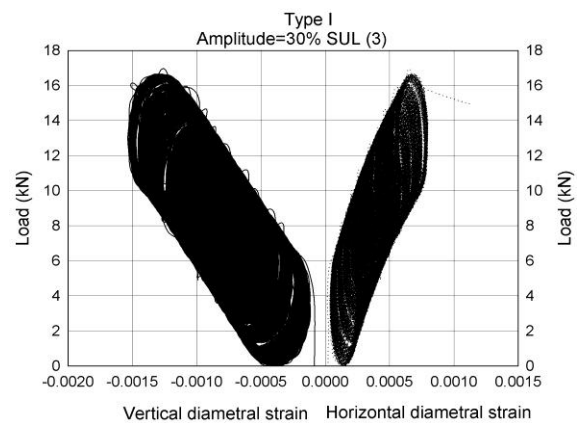
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(b)

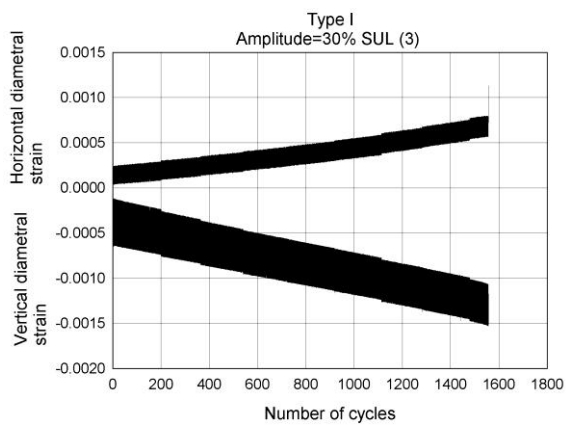


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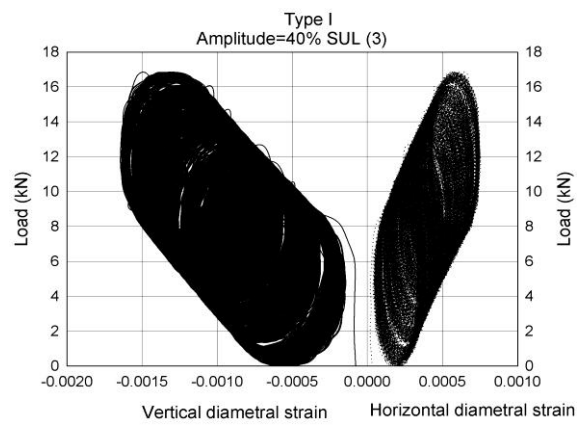


(c)

Figure A.2 Type I diametral cyclic loading;
amplitude = 4.6 kN (20% SUL) (a, b and c)

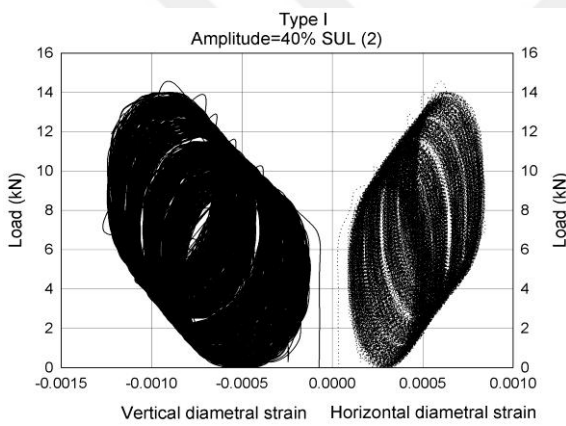


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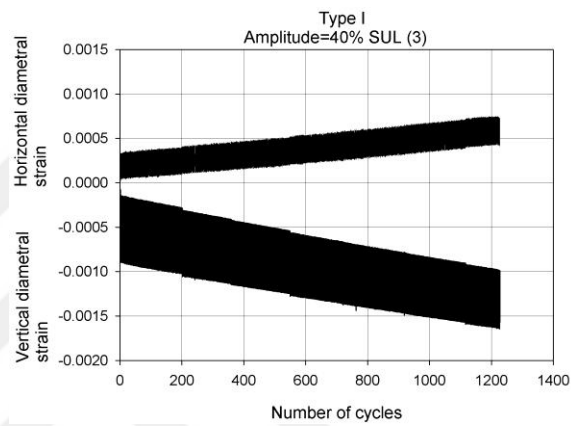


(c)

Figure A.3 Type I diametral cyclic loading;
amplitude = 6.9 kN (20% SUL) (a, b, c and d)

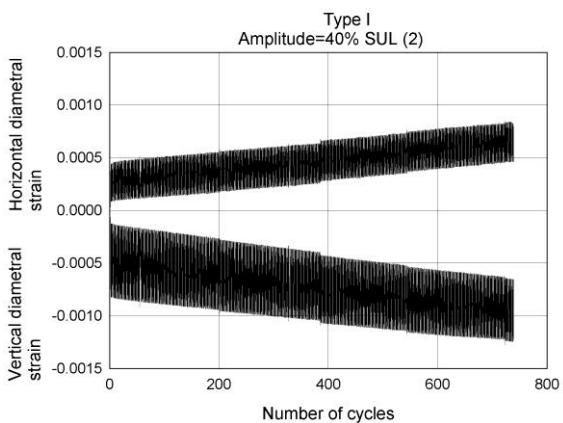


(a)



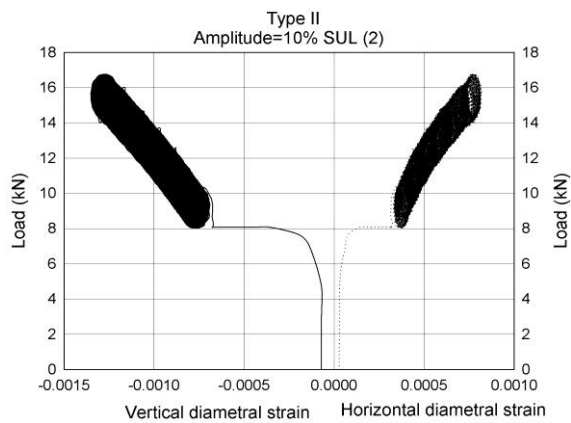
(d)

Figure A.4 Type I diametral cyclic loading;
amplitude = 6.9 kN (30% SUL) (a, b, c and d)

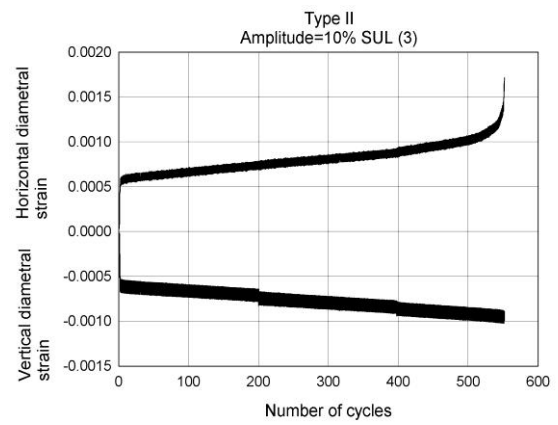


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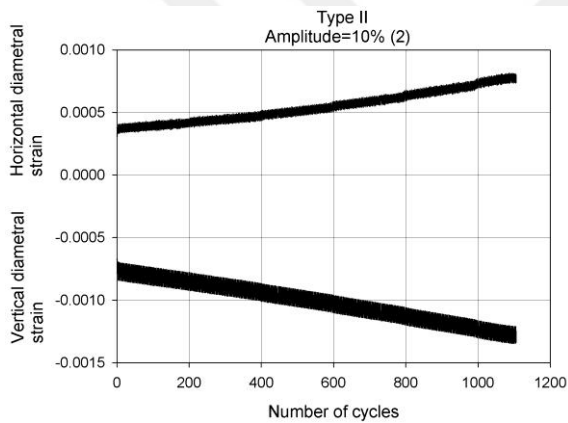
Effect of Type II cyclic loading on ITS of
Brisbane tuff



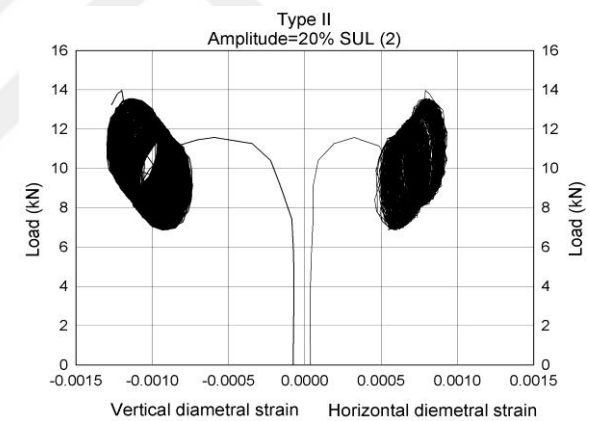
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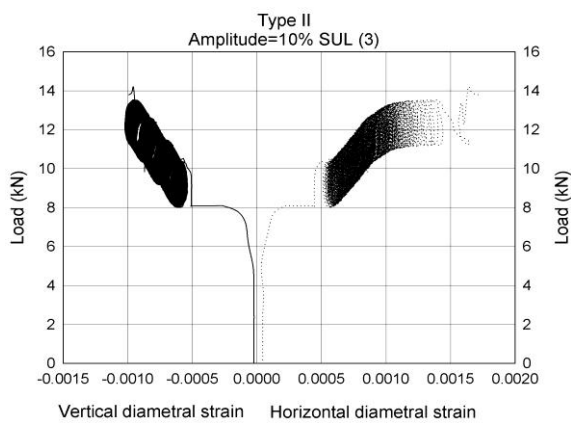
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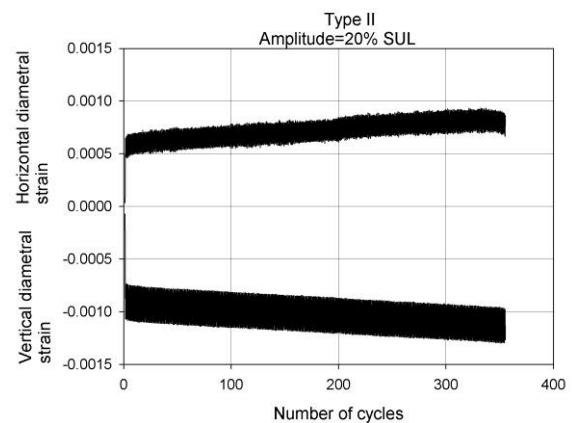
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(a)

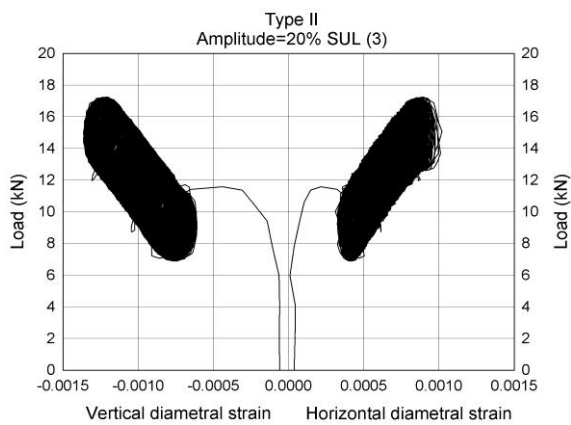


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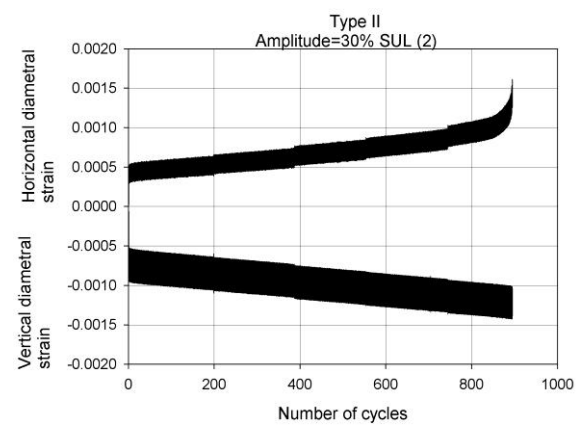


(b)

Figure A.5 Type II diametral cyclic loading;
amplitude = 2.3 kN (10% SUL) (a, b, c and d)

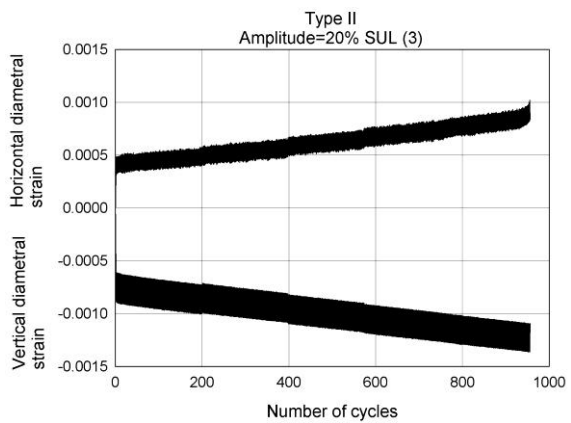


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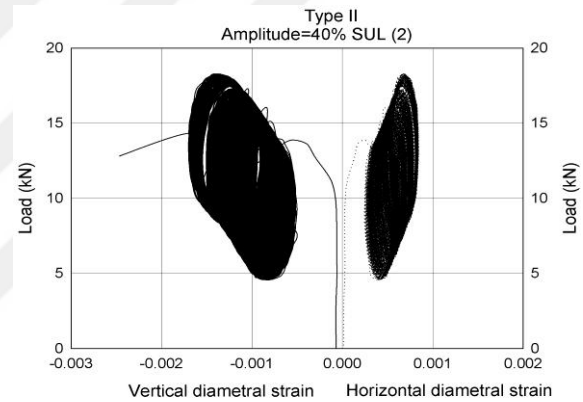
(b)

Figure A.7 Type II diametral cyclic loading;
amplitude = 6.9 kN (30% SUL) (a and b)

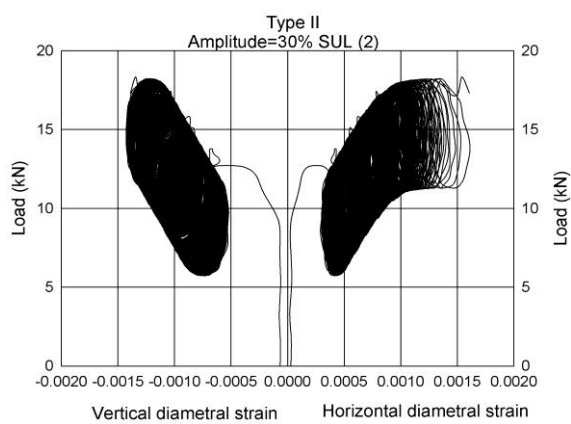


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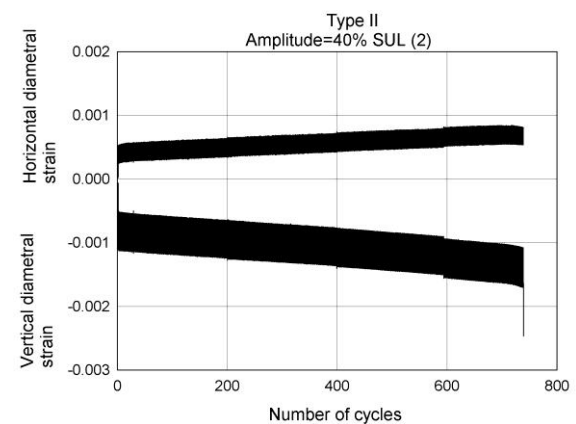
Figure A.6 Type II diametral cyclic loading;
amplitude = 4.6 kN (20% SUL) (a, b, c and d)



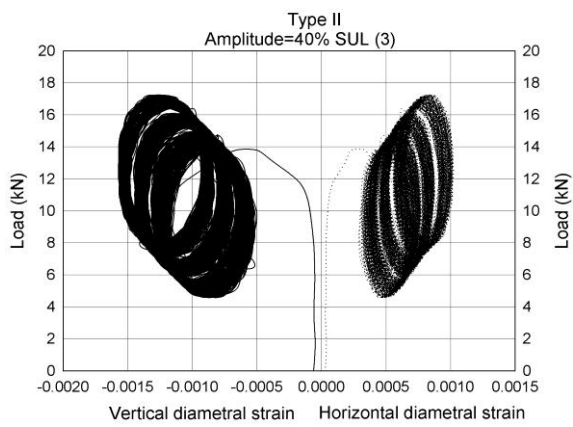
(a)



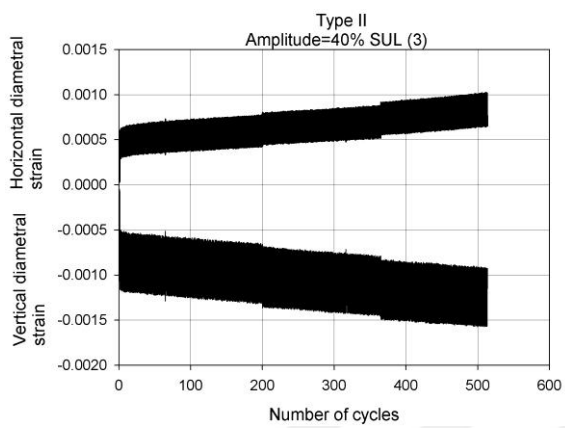
(a)



(b)



(c)



(d)

Figure A.8 Type II diametral cyclic loading;
amplitude = 9.2 kN (40% SUL) (a, b, c and d)

APPENDIX B

Degradation of Fracture Toughness of Rocks through Cyclic Loading

Effect of Same valley (Type I) cyclic loading on fracture toughness of Brisbane tuff

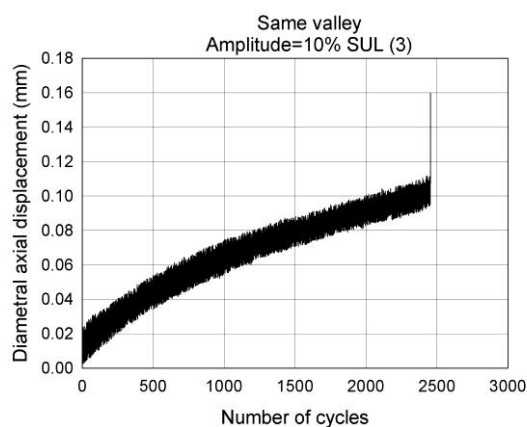
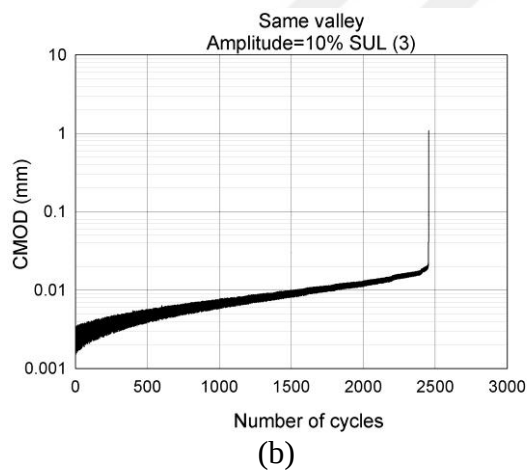
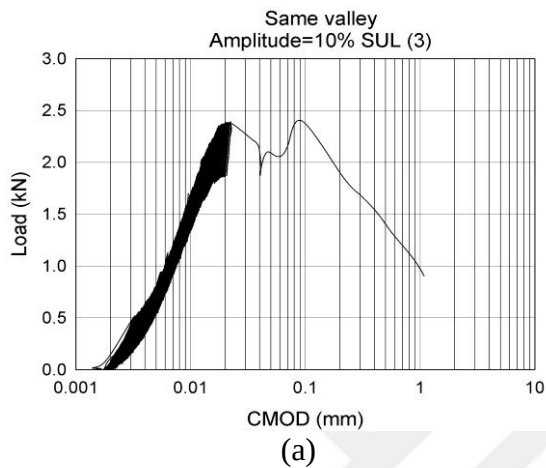


Figure B.1 Type I diametral cyclic loading; amplitude = 0.45 kN (10% SUL) (a, b and c)

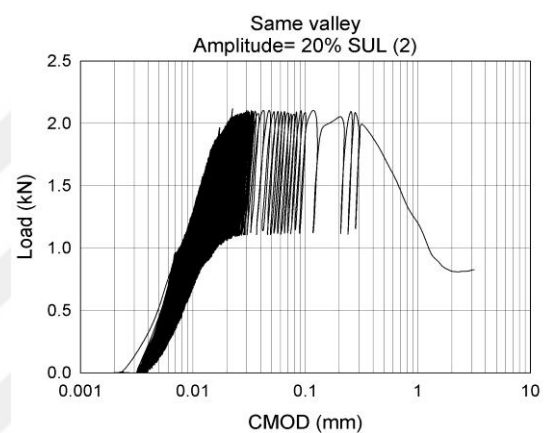


Figure B.2 Type I diametral cyclic loading; amplitude = 0.9 kN (20% SUL)

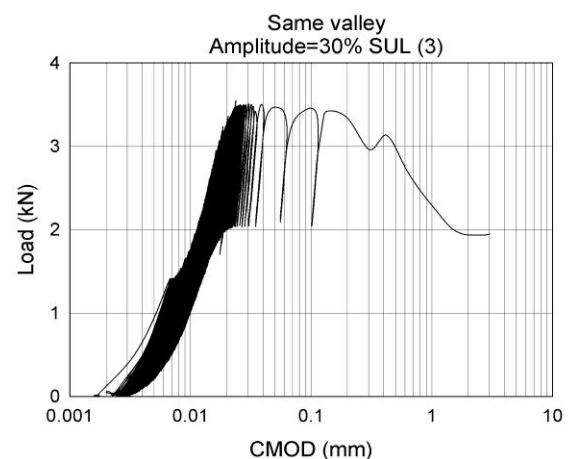
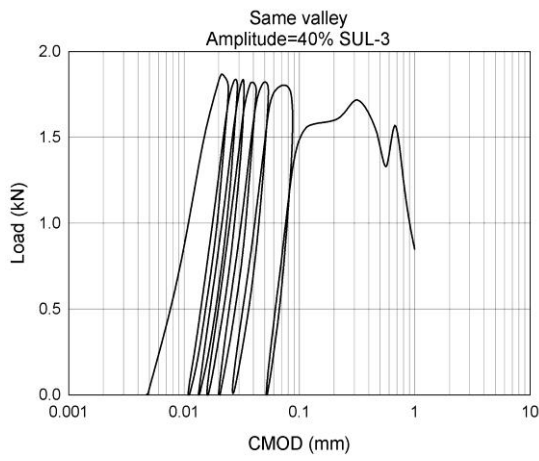
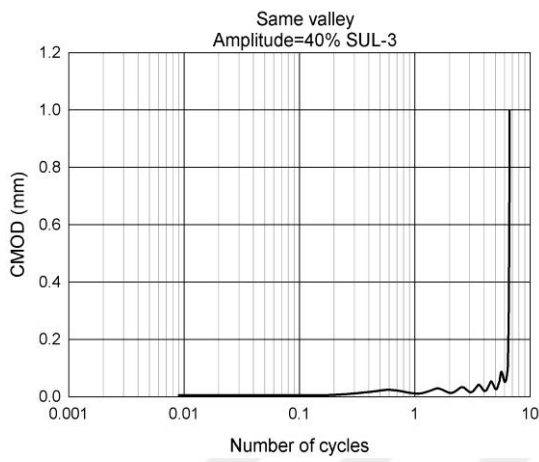


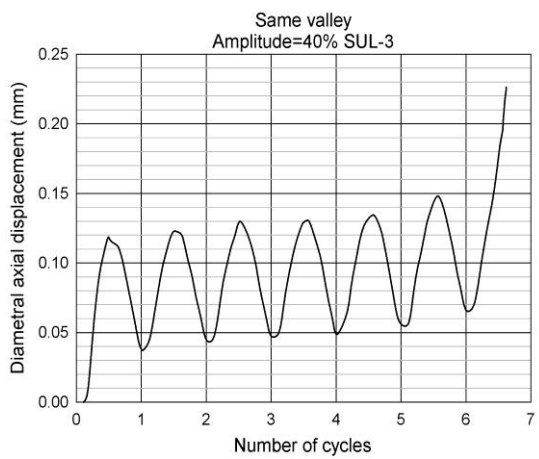
Figure B.3 Type I diametral cyclic loading; amplitude = 1.35 kN (30% SUL)



(a)

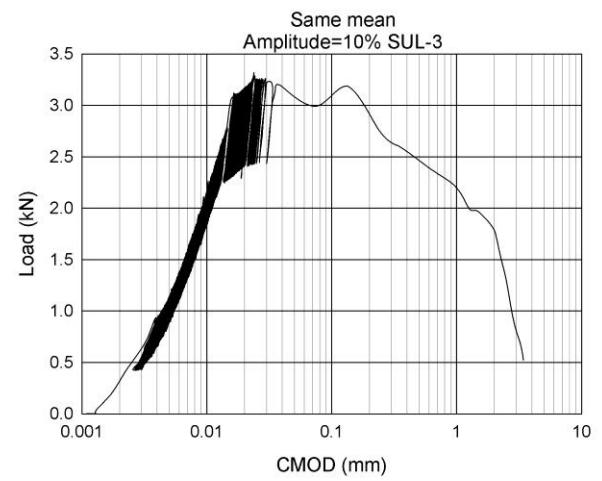


(b)

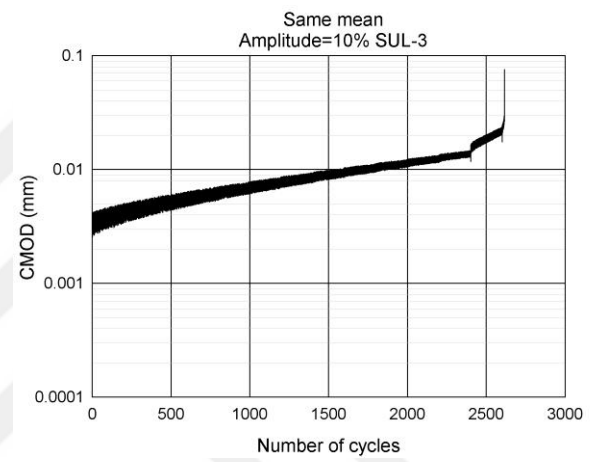


(c)

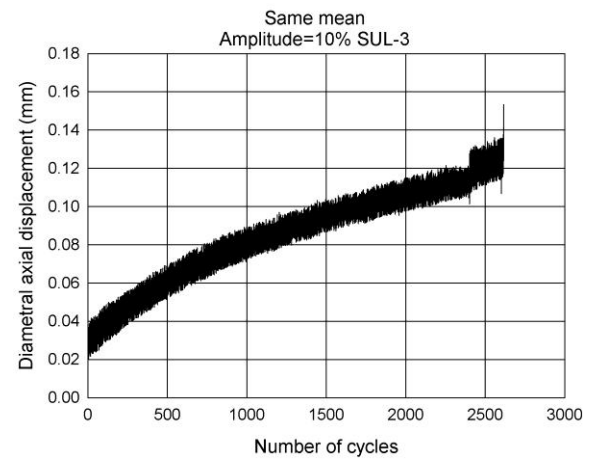
Figure B.4 Type I diametral cyclic loading;
amplitude = 1.8 kN (40% SUL)



(a)



(b)



(c)

Figure B.5 Type II diametral cyclic loading;
amplitude = 0.45 kN (10% SUL)

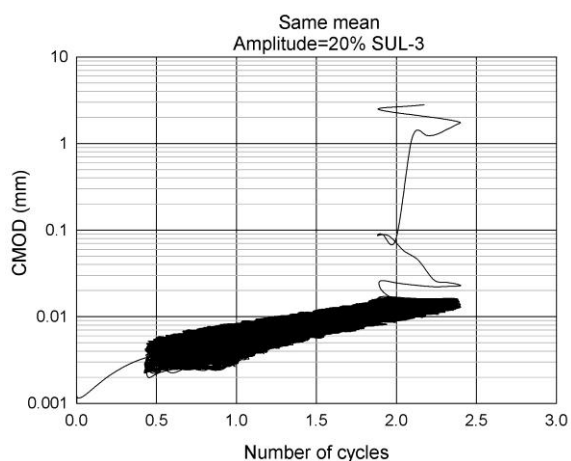
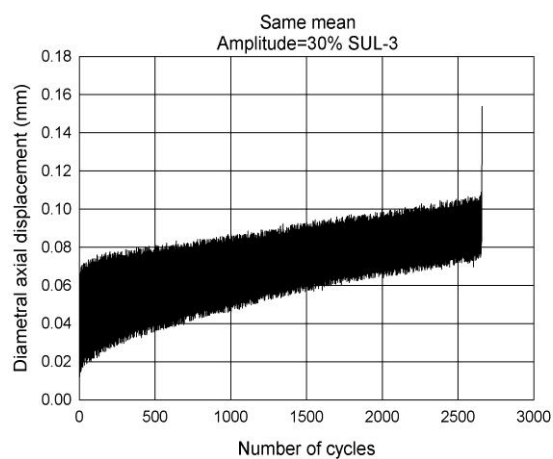
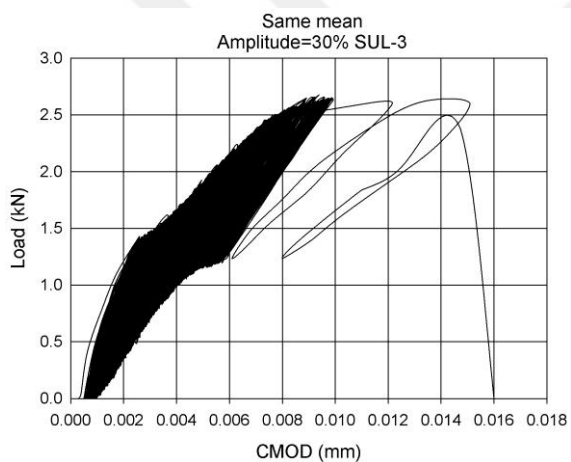


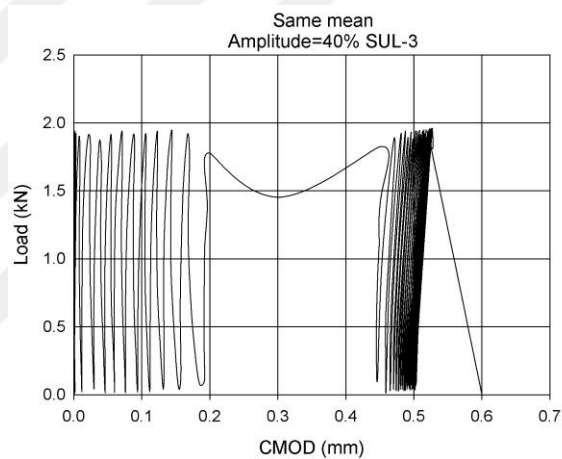
Figure B.6 Type II diametral cyclic loading;
amplitude = 0.9 kN (20% SUL)



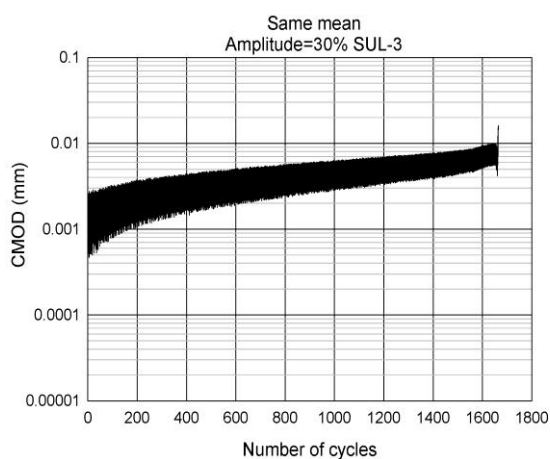
(c)
Figure B.7 Type II diametral cyclic loading;
amplitude = 1.35 kN (30% SUL)



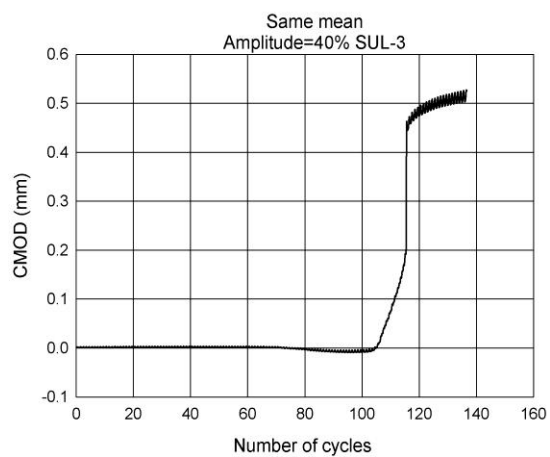
(a)



(a)



(b)



(b)

Figure B.8 Type II diametral cyclic loading;
amplitude = 1.8 kN (40% SUL)

APPENDIX C

X-Ray Diffractometer (XRD) Analysis

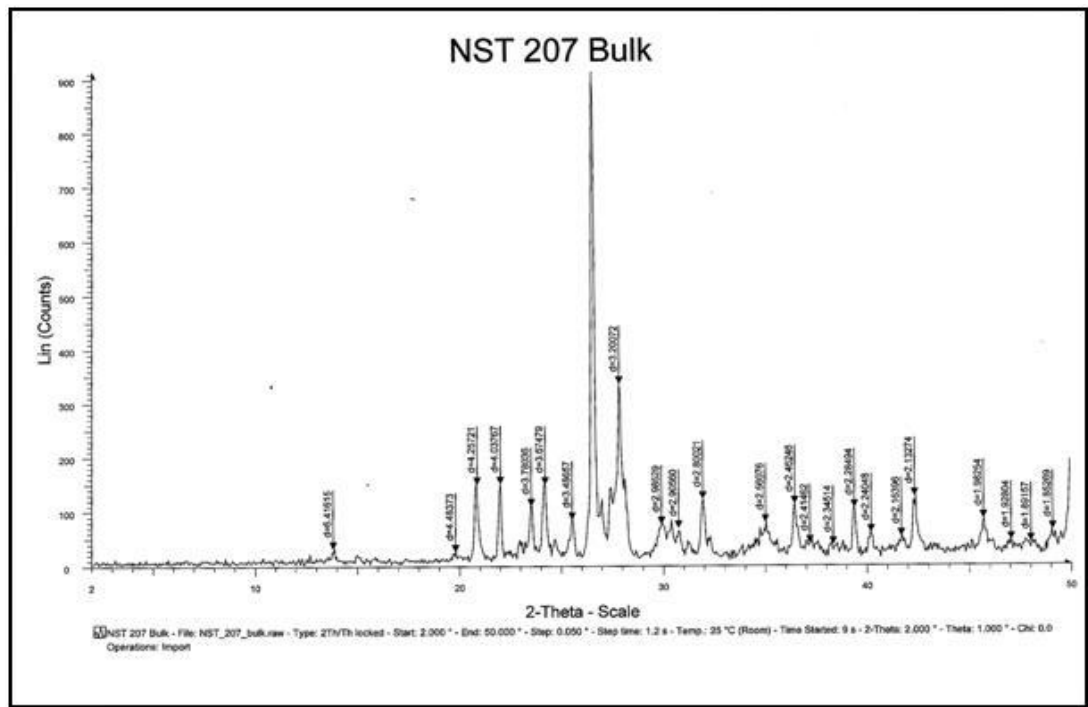


Figure C.1 XRD analyses of NST207 samples

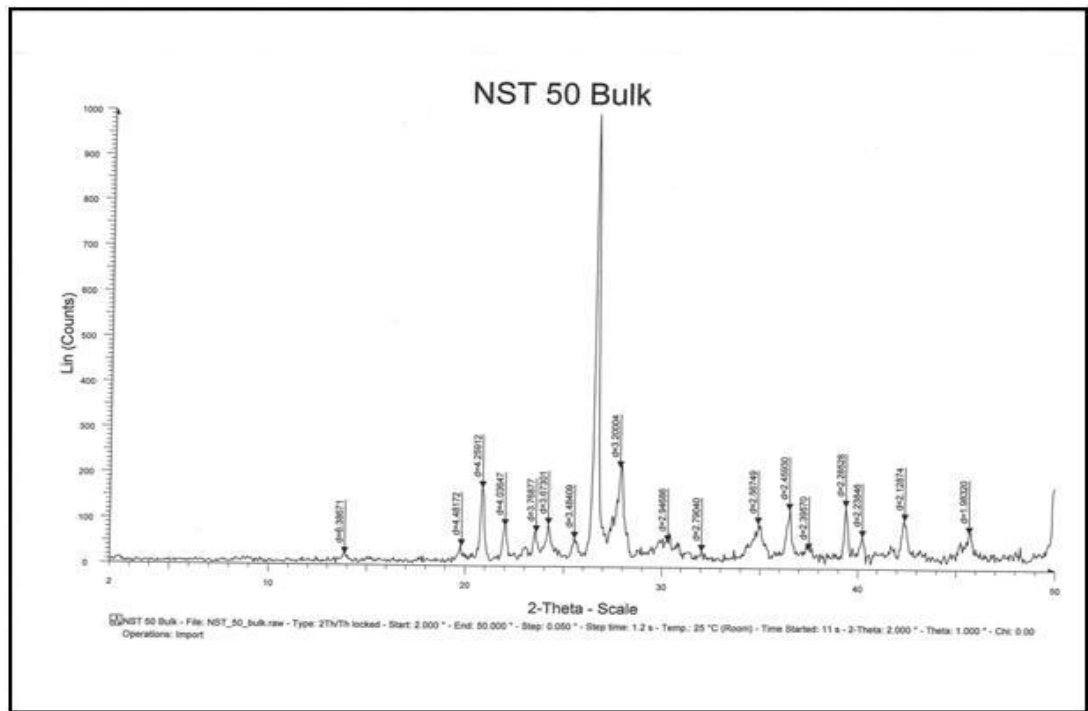


Figure C.2 XRD analyses of NST50 samples

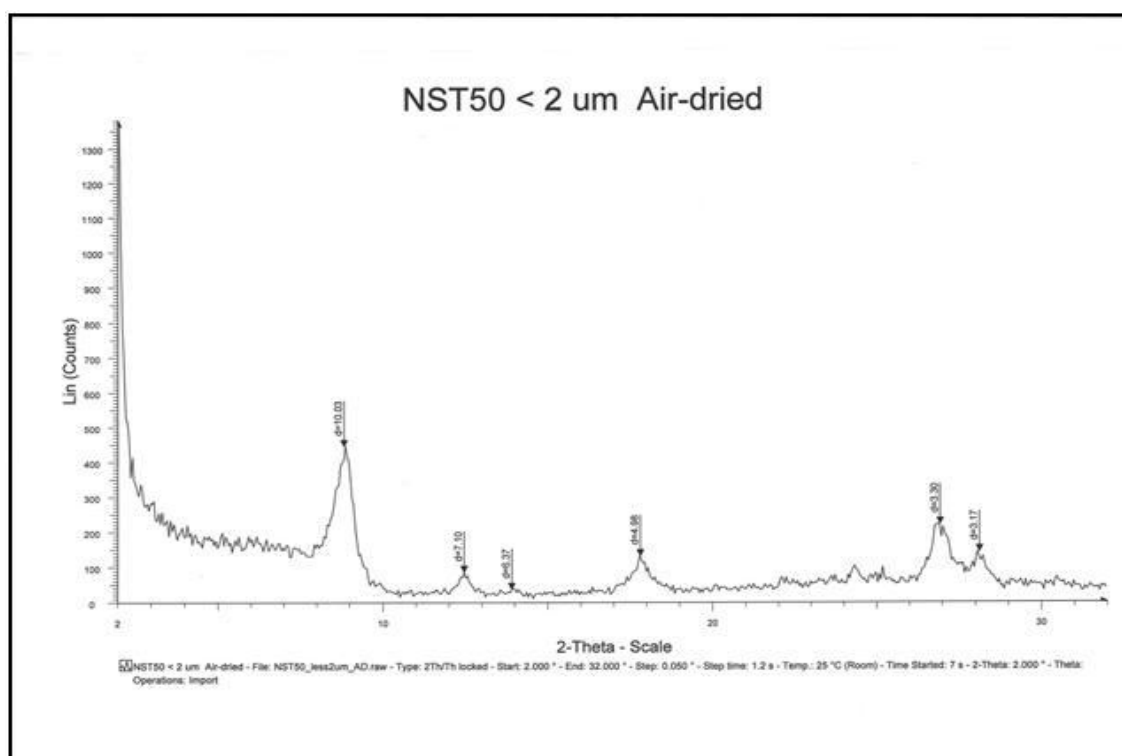


Figure C.3 XRD with $\text{CuK}\alpha$ radiation for detailed analysis of clay minerals in NST50 samples

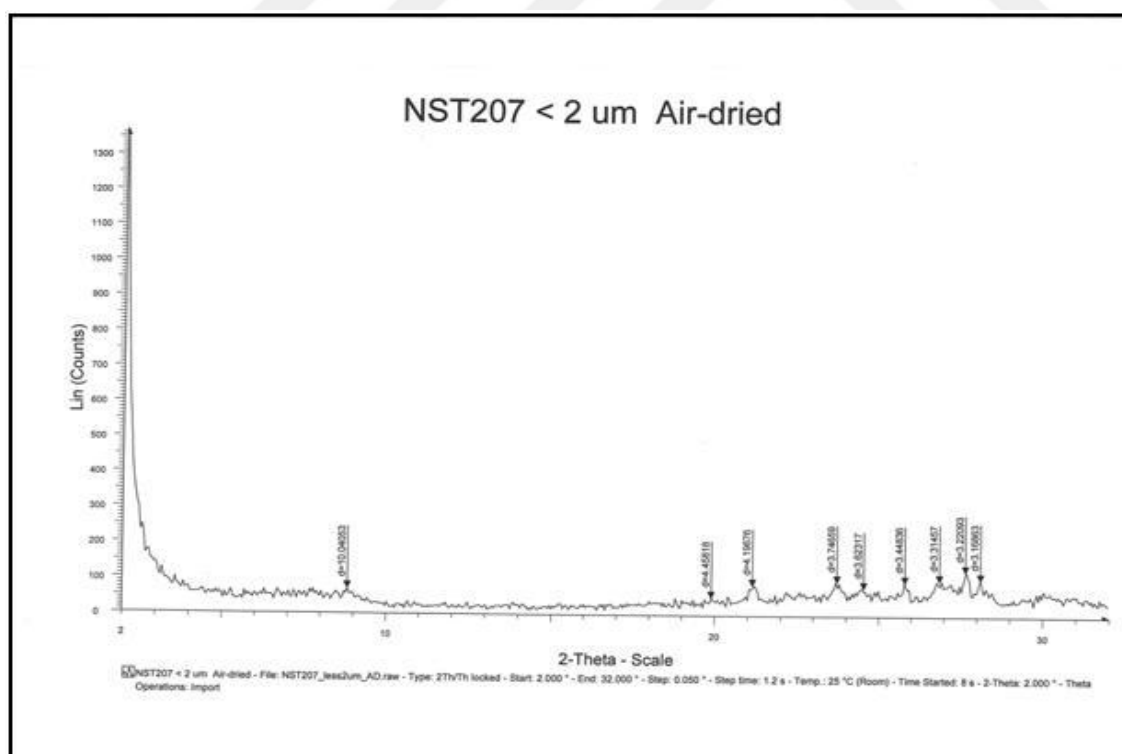


Figure C.4 XRD with $\text{CuK}\alpha$ radiation for detailed analysis of clay minerals in NST207 samples

APPENDIX D

SEM Images

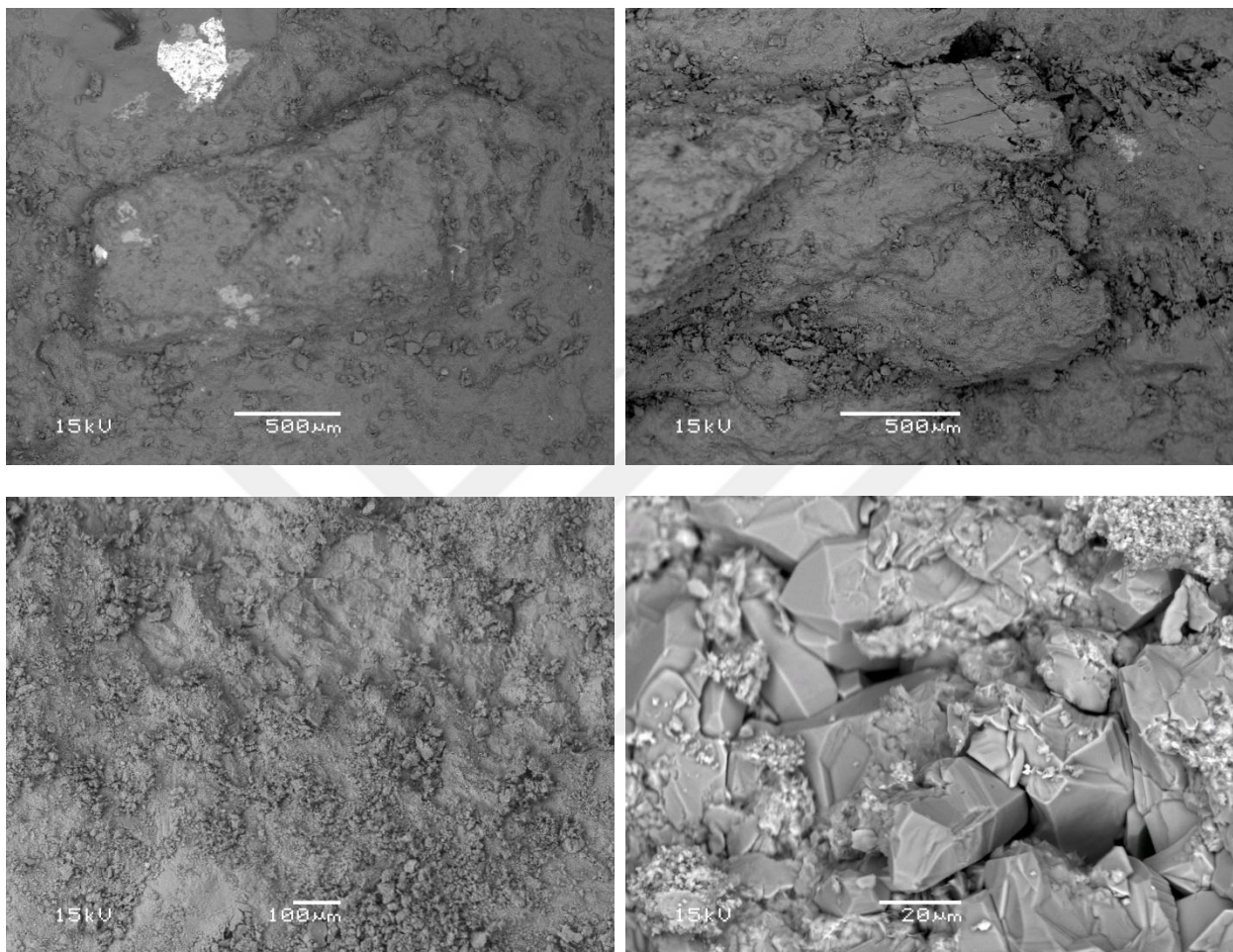
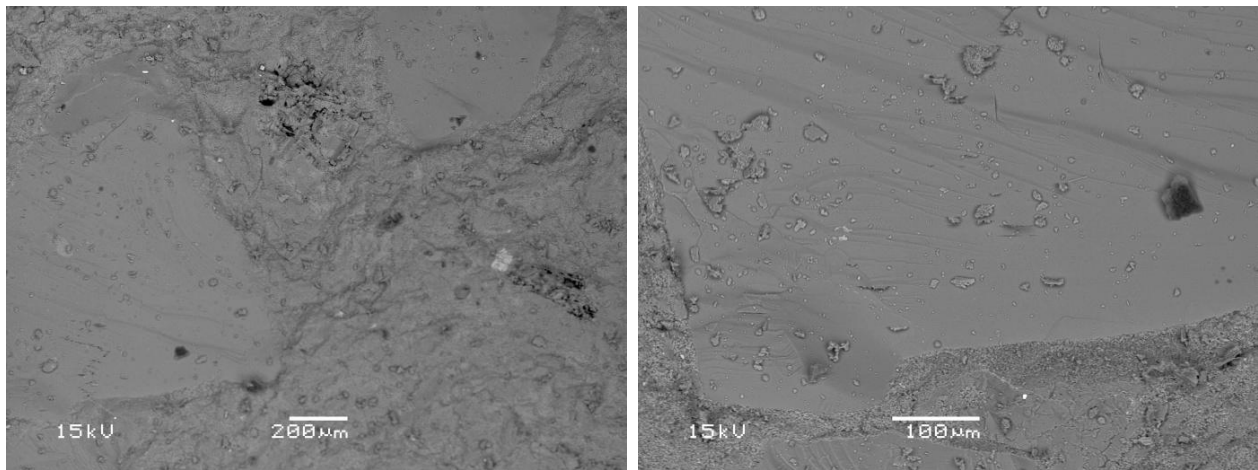


Figure D.1 Scanning electron microscope images of the fractured surface of Brisbane tuff disc specimens under cyclic loading



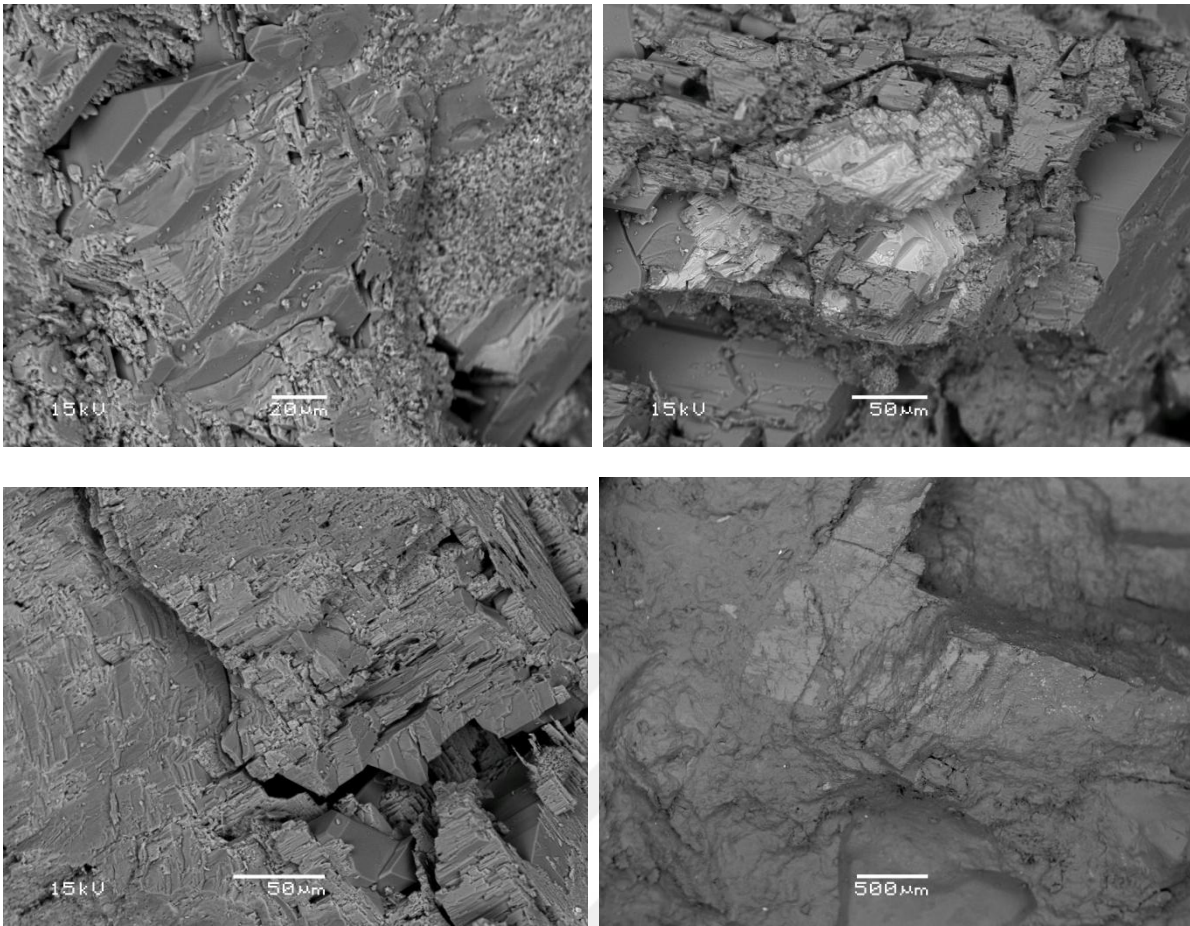


Figure D.2 Scanning electron microscope images of the fractured surface of Brisbane tuff disc specimens under monotonic loading

APPENDIX E

Doyens & Inspirers in my Research



Mustafa Kemal ATATURK (1881-1938)
The founder of the Turkish Republic
Turkish women have education rights because of him...



Prof David John WILLIAMS



Prof Ted BROWN



Emeritus Prof Charles FAIRHURST



Emeritus Prof John HUDSON



Dr Nick BARTON



Prof Peter KAISER