Study on the strength of jointed rocks with infill from South East Queensland

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Published
2023-04-12

Thesis Type
Thesis (PhD Doctorate)

School
School of Eng & Built Env

DOI
10.25904/1912/4818

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Study on the strength of jointed rocks with infill from South East Queensland

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Submitted in fulfilment of the requirements of the degree of
Doctor of Philosophy

December 2022
ABSTRACT

In geotechnical engineering practice related to mining, tunnelling, and oil exploration, a set of discontinuities might affect the overall strength and stability of rock mass. To address this issue, a good deal of research has been conducted in the past decades to study the behaviour of jointed rocks under different loading conditions. However, most of previous studies have been conducted on rock-like materials such as concrete or plaster, while only limited research has been done on natural rocks. This study seeks to bridge this knowledge gap and (1) to investigate the effects of discontinuity on the engineering properties of common rocks from South East Queensland, and (2) to develop a numerical model that can accurately estimate the strength of jointed rocks.

A series of unconfined compression tests on natural rocks of different geological origins (three types of sandstone, argillite, and basalt) were performed to better understand the effect of crack length (1 cm and 2 cm long) and infill material (sand or clay) on rock strength. In addition, a series of shear tests were conducted on jointed specimens of two types of sandstone with different infill thicknesses, ranging from 1mm to 3mm.

The obtained results from the laboratory tests were used to improve the current methods of rock shear strength predictions, which were initially designed to estimate the strength of artificial rock-like material. Also, a new numerical method was proposed to accurately estimate the unconfined compressive strength of rock specimens with pre-existing discontinuities filled with either clay or sand. This numerical model provided valuable insight into the failure mechanism of jointed rocks, which agreed with the data obtained in the laboratory.
By combining the experimental and numerical investigations, the mechanism of rock failure with different pre-existing cracks and infill material can be studied. The UCS values estimated by the numerical model and the damage model were able to estimate with a high degree of accuracy to the unconfined compressive strength of rocks with pre-existing discontinuities, and it allows to identify and explain the mechanisms of two common damage patterns observed in laboratory tests. The presence of discontinuities (1 cm or 2 cm long cracks) decreases the UCS of natural rocks, while filler material has a tendency to increase the overall strength of the tested rocks.
STATEMENT OF ORIGINALITY

This work has not previously been submitted for a degree or diploma in any university. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person except where due reference is made in the thesis itself.

(Signed) __________________________ Date  15/12/2022

Chen Cui
ACKNOWLEDGEMENT OF PAPERS INCLUDED IN THIS THESIS

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• Acknowledge all those who have contributed to the research, facilities or materials but who do not qualify as authors, such as research assistants, technical staff, and advisors on cultural or community knowledge. Obtain written consent to name individuals.

Included in this thesis are papers in Chapters 5, 6, 7 and 8 which are co-authored with other researchers. My contribution to each co-authored paper is outlined at the front of the relevant chapter. The bibliographic details for these papers including all authors, are:

Chapter 5:


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Chapter 6:

Chapter 7:


Chapter 8:


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Chen Cui

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Principal Supervisor: A/P Gratchev Ivan
ACKNOWLEDGEMENTS

On one day in December 2022, I finally finished my PhD thesis. Through these years I have spent on the study as a PhD candidate, I would like to give my deepest gratitude to my supervisor: A/Prof. Gratchev Ivan. I could not finish this journey without your patient, and your kindness. I also fully appreciate all of your contributions of time, valuable knowledge and experience. In addition, I also want to give my appreciation to my associate supervisor A/Prof Erwin oh, who also supported and encouraged me during this period.

Also, I am grateful for the support and funding from GUPRS and GUIPRS scholarships for my PhD study.

Many thanks go to the laboratory technicians at Griffith University School of Engineering. I would like to thank the help from Mr Juergen Zier and Mr Chuen Lo for their support and guidance with the laboratory experiments.

Last but not least, I would like to give my appreciation to my parents and friends who always supported me, and encouraged me to be better.
LIST OF PUBLICATION

The following publications are all peer-reviewed and produced from this research to disseminate the innovations and findings:

Published:


3 Gratchev, I., Ravindran, S., Kim, D.H., Cui, C., Tan Q.H. Mechanisms of shallow rainfall-induced landslides from Australia: insights into field and laboratory investigations. Progress in Landslide Research and Technology

Submitted:


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# Notations

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<tr>
<td>SE</td>
<td>Southeast</td>
</tr>
<tr>
<td>XRD</td>
<td>X-ray diffraction</td>
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<tr>
<td>QLD</td>
<td>Queensland</td>
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<td>PLI</td>
<td>Point load index</td>
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<td>UCS</td>
<td>Unconfined compressive strength</td>
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<tr>
<td>AS</td>
<td>Australian standard</td>
</tr>
<tr>
<td>MPa</td>
<td>Mega pascal</td>
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<td>GPa</td>
<td>Giga pascal</td>
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<td>S1</td>
<td>Sandstone 1</td>
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<td>S2</td>
<td>Sandstone 2</td>
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<tr>
<td>S3</td>
<td>Sandstone 3</td>
</tr>
<tr>
<td>PMMA</td>
<td>Polymethylmethacrylate</td>
</tr>
<tr>
<td>RMSE</td>
<td>Root mean square error</td>
</tr>
<tr>
<td>AE</td>
<td>Acoustic emission</td>
</tr>
<tr>
<td>CNL</td>
<td>Constant normal load</td>
</tr>
<tr>
<td>CNS</td>
<td>Constant normal stiffness</td>
</tr>
<tr>
<td>NSD</td>
<td>Normalized strength drop</td>
</tr>
<tr>
<td>RQD</td>
<td>Rock quality designation</td>
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<tr>
<td>JRC</td>
<td>Joint roughness coefficient</td>
</tr>
<tr>
<td>JCS</td>
<td>Unconfined compression stress of rock</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite element method</td>
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NOTATIONS

RFPA            rock failure process analysis code
DEM            discrete element method
PFC            particle flow modelling
UDEC            universal distinct element code

LIST OF SYMBOLS

\( r \)       rebound number on weathered joint surface
\( R \)       rebound number on unweathered joint surface
\( \varepsilon_{ij} \)   damage elastic strain tensor
\( \sigma_{ij} \)   stress tensor
\( E \)       Young’s modulus
\( E_0 \)       Young’s modulus
\( D \)       isotropic damage variable
\( V \)       Poisson's ratio
\( C_{(v)} \)   coefficient which related to Poisson’s ratio \( v \)
\( t_c \)       cohesive stress
\( d \)       atomic distance
\( \beta \)       decay coefficient
\( \tau \)       shear strength of jointed rock
\( \sigma_n \)   normal stress what applying on the jointed rock
\( \varphi_r \)   residual friction angle of rock
\( \alpha \)       orientation of the joint
\( \varphi_r \)   residual friction angle
\( \phi_b \)       basic friction angle
\( T_p \)       shear strength
NOTATIONS

f \quad \text{thickness of infill material.}

\mu \quad \text{shear strength (MPa).}

C \quad \text{experimentally derived constant}

m \quad \text{experimentally derived constant}

T_{\text{max}} \quad \text{maximum shear strength of unfilled joints}

T_{\text{min}} \quad \text{potential minimum shear strength}

\tau_p \quad \text{shear strength of infilled joint (MPa).}

\tau_0 \quad \text{shear strength of clean joint under normal load (MPa).}

\sigma_n \quad \text{applied normal strength (MPa)}

t \quad \text{infill thickness (mm)}

a \quad \text{average roughness of joint surface (mm).}

k_1 \quad \text{empirical constants}

k_2 \quad \text{empirical constants}

c \quad \text{empirical constants.}

\tau_{(h,CNS)} \quad \text{joint shear stress at a horizontal displacement of h}

K \quad \text{ratio of t/a and (t/a)_{cr}}

k_s \quad \text{t/a ratio over the critical t/a ratio.}

i \quad \text{initial asperity angle}

U_a \quad \text{pore air pressure which is normally considered as the normal air pressure}

m_{\text{sub}} \quad \text{saturated submerged mass}

m_{ss} \quad \text{saturated submerged mass of the basket plus sample}

m_{bs} \quad \text{saturated submerged mass of the basket alone}

m_{\text{sat}} \quad \text{saturated surface-dry mass}

m_{sd} \quad \text{saturated surface-dry mass of the sample plus container}
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m_c$</td>
<td>mass of the container</td>
</tr>
<tr>
<td>$m_g$</td>
<td>grain mass</td>
</tr>
<tr>
<td>$m_{cd}$</td>
<td>mass of the dried sample and container</td>
</tr>
<tr>
<td>$V_b$</td>
<td>bulk volume</td>
</tr>
<tr>
<td>$V_p$</td>
<td>pore volume</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>density of water at $t$</td>
</tr>
<tr>
<td>$t$</td>
<td>temperature</td>
</tr>
<tr>
<td>$n$</td>
<td>porosity of rock</td>
</tr>
<tr>
<td>$I_s$</td>
<td>uncorrected point load strength</td>
</tr>
<tr>
<td>$P$</td>
<td>load at failure</td>
</tr>
<tr>
<td>$D$</td>
<td>platen separation</td>
</tr>
<tr>
<td>$D_e$</td>
<td>equivalent core diameter</td>
</tr>
<tr>
<td>$I_{S(50)}$</td>
<td>point load strength index</td>
</tr>
<tr>
<td>$\theta$</td>
<td>rotation of block about the centroid</td>
</tr>
<tr>
<td>$x_i$</td>
<td>coordinates of block centroid</td>
</tr>
<tr>
<td>$u_i$</td>
<td>velocity components of block centroid</td>
</tr>
<tr>
<td>$t$</td>
<td>time</td>
</tr>
</tbody>
</table>
CHAPTER 1: INTRODUCTION

1.1 Research background

In geotechnical engineering, several factors significantly affect the strength of rock mass and the stiffness of heterogeneous brittle material like that found in the shear zone, dykes, major discontinuities and faults. For an intact rock, a few compositions can affect its strength such as voids, cracks and flaws. Even a tiny fracture can still decrease the strength of rock, and even the whole structure (Eberhard et al., 2004).

![Figure 1-1 Multiple set of flaws in rock slope on Gold Coast (Kim et al., 2015)](image)

Figure 1-1 Multiple set of flaws in rock slope on Gold Coast (Kim et al., 2015)

Figure 1-1 displays the open flaws on a rock slop. As a matter a fact, these kinds of pre-existing cracks as discontinuities are quite common. They manifest as phenomena such as the half-height of a mountain, a man-made damn, and sometimes even on the side of a highway.

Over the few decades, the propagation of cracking in rock has attracted considerable attention and substantial experimental research. Scientists studied a series of conditions of rocks with flaws. Many of these cases led to the conclusion that the initiation, propagation and coalescence of pre-existing flaws caused the failure (Eberhard et al., 2004; Cheng et al., 2016).
While cracks vary in size from microscopic to macroscopic, they all can induce the failure of rock during the whole tectonic motion process (Zhou et al., 2014). In previous research, uniaxial tests (Ramamurthy et al., 1994; Shen et al., 1995; Wong and Chau, 1998; Sagong and Bobet, 2002; Xu et al., 2013; Yin et al., 2014; Cao et al., 2015), biaxial tests (Bobet and Einstein, 1998), and shear box tests (Barton, 1976; Gehle and Kutter, 2003; Barton, 2013; Indraratna, 2010) have been conducted, by using rock-like material with multiple closed flaws or open flaws. Five types of cracks initiated by pre-existing flaws have been defined (Cheng et al., 2016):

- Wing crack
- Quasi-coplanar secondary crack
- Oblique secondary crack
- Out-of-plane tensile crack
- Out-of-plane shear crack

With this classification as a foundation, more and more different types of cracks have been discerned as propagating from these five types of cracks (Sagong and Bobet, 2002).

Previous studies show that in shear conditions, the crack propagation mostly initiates from a pre-existing flaw or flaws and grows into a rock bridge, eventually, this could produce a new fracture that would connect to the other pre-existing flaws (Gehle and Kutter, 2003). This process is quite similar to the mechanism of rock behaviours under shear and compressive loads. Using a multi-dimensional space method, researchers examined the initiation, propagation and coalescence of open flaws, experimentally and numerically tested for the effects of the crack process in a rectangular sample with a single open flaw, and explored the failure process of specimens with three flaws under uniaxial compressive load (Cheng and Zhou, 2014; Wong and Zhang, 2014; Tang et al., 2000). These studies of crack
propagation mostly involved rock-like material containing different numbers of opened or closed flaws and they indicated that the new cracks started near the pre-existing flaws under compressive load and proceeded along the axial stress to another pre-existing flaw or directly caused the failure of the sample (Li et al., 2005; Park and Bobet, 2009; Yang et al., 2012; Cheng et al., 2016).

The effects of unconfined compressive tests on five different types of rock samples with open flaws are investigated in this study. This is significant as rock behaviour with infill in the flaw under compressive load is rarely studied. In this paper, the effect of such infill is studied. In all the experiments, the specimens were obtained from the Gold Coast area, and it is the same with the natural sand and clay filling.

In addition, some of the characteristics that would also affect the rock slope strength behaviour are discontinuities in rock slopes, such as joints, bedding planes, and faults (Figure 1-2). Of these, jointed discontinuity is crucial to shear behaviour, and this is the most common cause of rock mass failures (Patton, 1966; Barton, 1973; Hoek and Brown, 1980; Gratchev and Kim, 2013; Raghuvanshi, 2019; Cui et al., 2019).

![Figure 1-2 The potential rock slope failure (Raghuvanshi 2019)](image_url)
In natural conditions, usually the rock joints are filled with natural materials such as friction material (e.g., sand) and cohesive material (e.g., clay). To study the influence of jointed discontinuity on the rock mass strength, these related factors have been studied: the compressive strength of rock, the applied normal stress, the degree of joint roughness, the type and thickness of the infill material, the joint spacing and separation, the joint size, and also the degree of saturation (Barton, 1973; Sinha and Singh, 2000; Jahanian and Sadaghiani, 2015; Hencher and Richards, 2015; Ram and Basu, 2019; Zhao et al., 2020; Bhardwaj and Rao, 2022; Wu et al., 2022). Laboratory tests have been carried out to study the effect of these different factors with and without various infilled materials. Based on the difference in the loading conditions, two types of experiments have been conducted: those of the constant normal load (CNL) condition and those of the constant normal stiffness (CNS) condition (Goodman, 1970; Kanji, 1974; Lama, 1978; Phien-Wej et al., 1990; Papalianges et al., 1993; De Toledo et al., 1993; Indraratna et al., 1999, 2005; Indraratna and Welidniya, 2003; Brown, 2004; Jahanian and Sadaghiani, 2015; Cheng et al., 2016; Karakus et al., 2016; Lu et al., 2017; Zhao et al., 2021; Kasyap and Senetakis, 2022). It is implied that the shear behaviour of the infilled joints is correlated to the surface geometry, the type of infill material and the ratio of the infill thickness and the asperity height \((t/a)\) (Phien-Wej et al., 1990; Indraratna et al., 2005, 2008; Oliveira et al., 2009). In light of these results, it has been ascertained that when the infill is thicker, the shear strength of the joint is lower, with less friction of the joint surface and more cohesive infill material.
Figure 1-3 Cross-section of rock joint with infill: (a) saw-tooth model with infill, (b) natural core rock joint

Figure 1-3 shows a model cross-section of a saw-tooth infilled rock joint, which was widely used in previous research. The ratio of the infill thickness and the asperity height was based on this type of rock joint model. When the t/a is equal to or less than 1, shear mostly happens between the joints and the thin fills (Indraratna and Mylvaganam, 2005). In the t/a ratio range of 0.35-0.72, the shear strength of the rock joints decreased (Lama, 1978). Especially when the t/a is between 0.07 and 0.25, the strength of rock joints can drop by as much as approximately 50%. Goodman (1970) and De Toledo et al., (1993) were of the view that the reason the shear strength is higher when the t/a is greater than 1.25, is because the
shear behaviour only occurs between the infill materials. However, Jahanian and Sadaghiani (2015) published results which indicated that only when the t/a ratio is over 2.0, will there be no intersection between the shear and the rock joints.

In the saw-tooth infilled rock joint model, the joint surface is artificial, unlike natural rocks. It is easier to determine the t/a ratio and more difficult to determine the exact asperity height for a saw-tooth model compared with a natural rock joint with various joint roughnesses. However, the jointed roughness coefficient is easier to access than the asperity height, not only in field work but also in lab testing. Besides, plaster or concrete as rock-like material has been widely used. However, an inhomogeneous material generally does not behave the same as a homogeneous material. In this study, the relationship between the joint roughness coefficient and the infill thickness, together with the influence on the shear behaviour of natural jointed rock is.

1.2 Research gap

- Studies on real rocks: Most studies or crack propagation have been performed on various types of uniform rock-like material under unconfined compression tests. As there is limited research on real rocks, it is not clear whether this research can be applied to real rocks because not generally being uniform, they have different properties. The crack propagation of natural rock samples has only been studied under shear tests. Given the limited studies on the crack propagation of natural rock samples, the question for my research is whether there is some difference in crack propagation when the sample is a natural rock, and if so, what these differences may be.

- Filled cracks in rocks: The influence of opened cracks only on the rock sample under an unconfined compressive test has not been totally investigated, not to mention if there is
an infill inside. At present no studies exist on the infill’s influence on rock strength under compressive tests, nor on the infill’s influence on crack propagation.

- Rock from the Gold Coast area. Rock types and their properties vary with area. Limited research has been conducted on rocks from the Gold Coast area, which makes it difficult for engineers to determine the properties of such rocks. The present study includes some experiments to investigate if rocks from different areas of the Gold Coast have different failure patterns and strengths, according to the mineral composition.

- For most studies, no proper methodology exists to inform the construction of a constitutive model. Such a model can be used to describe the effect of infill and cracks on rock properties using a numerical approach. This will be attempted in this study.

1.3 Research objective

The main objective of this research is to better understand the influence of different infill materials on normal and shear stress. This will be achieved by identifying the major influences caused by the different geometries of discontinuities, based on various lab tests and numerical modelling. The objectives of this study are:

1) To study the effects of discontinuity on the engineering properties of common rocks from SEQ, which includes
   - To study the effects of discontinuity size and infill on the unconfined compressive strength of natural rocks.
   - To investigate the influence of discontinuities with infill on the shear strength of jointed natural rocks.

2) To develop a constitutive model which can accurately predict the effect of discontinuities and infill material on the strength of natural rocks.
1.4 Scope of thesis

With the varieties of discontinuities existing in the structure of natural rock, it is important to clarify the scope of this study:

- The main purpose of this study is to investigate the behaviour of some common rock samples collected from the Gold Coast, in Queensland, Australia.
- This study focuses on the influence of the 1 cm or 2 cm crack and the sand or clay infill of the discontinuity.
- A constitutive numerical model is tested based on the different results from the laboratory work. This model can be suitable for other conditions.

1.5 Layout of thesis

This thesis contains the following chapters:

- Chapter 1: Introduction

  This chapter provides basic information on the background and introduces the research gaps, objectives and scope of this study. It concludes with an outline of the thesis structure.

- Chapter 2: Literature review

  A list of related studies is discussed in this chapter. It consists of a review of the existing research, its methodology and contributions. This review helps to identify research gaps that are addressed in this thesis.

- Chapter 3: Methodology

  The method of this study is discussed in this chapter which gives comprehensive details of the procedure that has been used in this research.
• Chapter 4: Geology study
  This chapter constitutes a discussion of the material used in this research.
• Chapter 5: Effects of cracks and infillings on strength of natural rocks.
  This chapter contains the results and discussion about the behaviour of rocks under uniaxial stress with different geometries of cracks and different materials and thicknesses of the infill. The chapter is based on a published paper “Effects of pre-existing cracks and infillings on strength of natural rocks: Cases of sandstone, argillite and basalt” which appeared in the Journal of Rock Mechanics and Geotechnical Engineering in 2020, authored by Cui and Gratchev.
• Chapter 6: Effect of joints on sandstone and argillite.
  This chapter is based on a publication in the International Journal of Geomate, “Changes in joint surface roughness of two natural rocks during shearing” by Cui et al, which appeared in 2019. In this chapter, the joint roughnesses of different rock samples are investigated. It includes a discussion of the behaviour of different samples under various normal stresses. A modified damage factor is proposed for different types of rock samples.
• Chapter 7: Effect of joints on sandstone and argillite
  The behaviour of a filled jointed sample is discussed in this chapter. This highlights the relationships between the infilled material and joint roughness on the shear stress. This chapter has been submitted as “The effect of clay infill on strength of jointed sandstone: laboratory and numerical analysis” by Cui and Gratchev 2022 in the Journal of Rock Mechanics and Geotechnical Engineering.
• Chapter 8 Numerical analysis of the effect of pre-existing cracks with infill on the strength of natural rocks
This chapter describes the process of the numerical model building up. This chapter is based on the submitted paper “Cui and Gratchev (2022). Numerical analysis of the effect of pre-existing cracks with infill on the strength of natural rocks. Rock Mechanics and Rock Engineer. Under review”.

- Chapter 9: Discussion and conclusion remarks

  This chapter shows the general information and conclusions for my PhD study.

- References

- Appendix
CHAPTER 2: LITERATURE REVIEW

This chapter offers a summary of the relevant research previously conducted. It consists of two main parts. The first is an introduction to research on unconfined compressive testing. The second covers research into the shear strength of rocks.

2.1 Crack propagation studies under compressive load

Previous research has mostly focused on crack propagation and the strength of rock-like material. The purpose of this section is mainly to demonstrate the previous research for crack propagation and the study by different numbers and geometries of pre-existing cracks.

2.1.1 Intact sample study

To find a more efficient method to monitor and track the rock crack propagation during the failure process, Liu and others (2006) implemented a series of experiments by using real-time holographic interference to continuously and dynamically monitor the propagation and deformation process of rock cracks under uniaxial and compression shear loads. Based on a quantitative analysis of the dynamic stripes, the mechanical factors of the rock crack failure propagation process was obtained.

These researchers used real-time holographic interference method based on a continuous wave laser as a lighting source and a holographic dry plate as a medium to record a dynamic interference fringe which can present the different strains of the various rock surface (Figure 2-1).
The experiment effectively identified the mechanical property of the rock crack propagation, for the rock sample under compressive and shear stress. The propagation originally cracks from the “I” type, and with a higher loading or change of direction, the crack propagation could change to “I-II” or “I-II-III” complex cracks. Besides, with the development of the stress of the material, the crack also varied. This could induce uniform forces or stress, which would cause some new open and closed crack.

This experiment accurately presented crack development and provided variates for calculating off-plane displacement and deformation. However, this method is not suitable for large deformations, it only can work on partial and small deformations. If the size of the sample range is above that suitable for measurement by holographic interference, this method needs to work on a section-by-section basis of the specimen instead of the whole specimen.

2.1.2 One pre-existing crack

Experimental testing for the deformation of the sample and numerical modelling for the sample strain changes in experiments have been widely performed in the past.

Xu et al. (2013) tested the different geometries of partially jointed rocks in terms of joint location, orientation, and trace length. Table 2-1 and Figure 2-2 show the different variations of the geometry, and the basic parameters of the numerical specimen. As can be
seem in the Figure 2-2, the length of the cracks are various from 33.6 mm to 126 mm, and the angle of joint is 45°, 60°, and 90°.

**Table 2-1 Mechanical parameters of numerical specimen (Xu et al., 2013)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Rock block</th>
<th>Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>Homogeneity index</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>Mean elastic modulus (GPa)</td>
<td>35</td>
<td>5</td>
</tr>
<tr>
<td>Mean axial strength (MPa)</td>
<td>130</td>
<td>30</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.25</td>
<td>0.2</td>
</tr>
<tr>
<td>Angle of internal friction (°)</td>
<td>38</td>
<td>30</td>
</tr>
<tr>
<td>Ratio of tensile to compressive strength</td>
<td>0.1</td>
<td>0.01</td>
</tr>
</tbody>
</table>

The numerical model used in this study was built by rock failure process analysis code (RFPA²D). The model behaviour should follow constitutive law which is for the isotropic and elastic material at instantaneous loading (Xu et al., 2013). This is presented as:

\[
\varepsilon_{ij} = \frac{1 + v}{E} \sigma_{ij} - v \frac{E}{E_0} \delta_{ij} \sigma_{ij} \delta_{ij} 
\]

\[
E = E_0 (1 - D) 
\]

Where:

\(\varepsilon_{ij}\): damage elastic strain tensor

\(\sigma_{ij}\): stress tensor

\(E\) and \(E_0\): Young’s modulus

\(D\): isotropic damage variable

\(V\): Poisson's ratio

In their study, Xu and others argued that in the case of uniaxial state stress, the equation can be rewritten to:

\[
\sigma_{11} = E_0 (1 - D) \varepsilon_{11} 
\]
CHAPTER 2

Figure 2- 2 Different geometries of experiments by (a) location, (b) orientation, (c) trace length (Xu et al., 2013)

This takes into account the longitudinal stress and strain components only. Therefore, the model is highly dependent on the damage variable D.
Figure 2-3 Comparisons between experimental and numerical results on failure stress in three different geometry (Xu et al., 2013)

Besides the differences of comparisons stress shown in Figure 2-3, the failure patterns in crack propagation for this material in differential geometries were investigated (Xu et al., 2013). The failure patterns and strain distributions in the experiment and numerical modelling were compared.

Figure 2-4 Failure pattern and strain distribution of experiment and numerical model in various joint location with different joint length (Xu et al., 2013)
From Figure 2-4 to 2-6, the numerical results not only verified the experimental data, but also proved the strain varying the failure procedure. In conclusion, based on the result of the joint location, more energy for a failure by rupture is required for a longer specimen. This means that with the same joint location, a longer specimen needs higher stress to cause the sample’s failure. In terms of the obtained results concerning the joint orientation, the relationship obtained from the fully spanning joints cannot directly work for the partially spanning joints. The numerical model for the variety of joint trace lengths suggests there is a linear relationship for the compressive strength of the partially cut specimens in terms of...
their joint trace lengths. Moreover, the joint trace length has a great influence on the stress-strain curves for samples with a partially spanning joint.

<table>
<thead>
<tr>
<th>JTL ratio</th>
<th>0.4</th>
<th>0.6</th>
<th>0.8</th>
<th>1.0</th>
<th>1.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure pattern</td>
<td><img src="image1.png" alt="Image" /></td>
<td><img src="image2.png" alt="Image" /></td>
<td><img src="image3.png" alt="Image" /></td>
<td><img src="image4.png" alt="Image" /></td>
<td><img src="image5.png" alt="Image" /></td>
</tr>
<tr>
<td>Numerical results</td>
<td><img src="image6.png" alt="Image" /></td>
<td><img src="image7.png" alt="Image" /></td>
<td><img src="image8.png" alt="Image" /></td>
<td><img src="image9.png" alt="Image" /></td>
<td><img src="image10.png" alt="Image" /></td>
</tr>
<tr>
<td>ARAMIS results</td>
<td><img src="image11.png" alt="Image" /></td>
<td><img src="image12.png" alt="Image" /></td>
<td><img src="image13.png" alt="Image" /></td>
<td><img src="image14.png" alt="Image" /></td>
<td><img src="image15.png" alt="Image" /></td>
</tr>
<tr>
<td>Strain distribution</td>
<td><img src="image16.png" alt="Image" /></td>
<td><img src="image17.png" alt="Image" /></td>
<td><img src="image18.png" alt="Image" /></td>
<td><img src="image19.png" alt="Image" /></td>
<td><img src="image20.png" alt="Image" /></td>
</tr>
</tbody>
</table>

Figure 2-6 Failure pattern of experiment and numerical in various joint trace length ratios with different JTL ratio (Xu et al., 2013)

A similar numerical study has been conducted by Wong et al. (2014) using a different model. These researchers used a bonded particle model, which is one of the distinct element methods, and compared this with the testing results when using gypsum samples (Wong and Einstein, 2009; Zhang and Wong, 2012). The 3-D geometry is shown in Figure 2-7.
The main stress intensity factor (Eringen, 1977) for this type of numerical model can be presented as:

$$K_1 = \frac{1}{\sqrt{2}C(v)} \tau_c \sqrt{\pi d}$$

Where:

- $C(v)$: coefficient which related to Poisson’s ratio $v$
- $\tau_c$: cohesive stress
- $d$: atomic distance

The length of the flaw is represented by $2a$ and the ratio of the particle radius and specimen size ($R/D$) is the specimen’s resolution. $\beta$ is the flaw’s inclination angle to the horizontal level. Based on previous studies (Potyondy and Cundall, 2004; Fakhimi and Villegas, 2007; Koyama and Jing, 2007), the main influence on rock strength in this kind of situation is exerted by the particle size and specimen size. From the testing data obtained by Wong (2013), the flaw inclination angle does not make a significant difference. Therefore, the crack resolution $\psi$, which is the ratio of $a$ and $2R$, represents the variations of the crack.
CHAPTER 2

Through numerical modelling, it was found that the uniaxial compressive strength is almost irrelevant to the crack resolution (ψ), but crack propagation is influential. This modelling showed that, with the decrease of crack resolution, an increase appears in the first initiation of stress and the time at which cracks initiated.

2.1.3 Two pre-existing cracks

2.1.3.1 Two pre-existing cracks with different bridge angle

In 2014, Yin and others performed a series of experiments to examine the coalescence mechanism between two parallel 3-D pre-existing surface cracks in granite under uniaxial compression. The two pre-existing cracks of the bridge angles varied from 0° to 135°, as is illustrated in the geometry shown in Figure 2-8. The digital speckle correlation method (DSCN) was used to analyse the capture images producing strain fields during the cracking process. During this experiment, several types of cracks were discovered: white patches, wing cracks and anti-wing cracks. The wing and anti-wing cracks comprised several sub-types of cracks, although differences were attributed to different observation angles or modes of analysis.

Figure 2-8 Sample geometry (Yin et al., 2014)
In Figure 2-9, (a) is a representation of a wing crack, (b) and (c) are Ts mode anti-wing cracks. Panels from (d) to (f) depict various sizes of cracking ranging from micro cracking to macro cracking, and (g) shows white patches. Therefore, these summarize two main types of surface crack: wing cracks and anti-wing cracks. Figure 2-10 shows the propagation of crack patterns observed in this study.
Figure 2-10 Coalescence patterns summarized in Yin et al., 2014

Based on the DSCN method for investigating coalescence (β at 0°, 46°, 60° and 90°), Yin and others drew a few conclusions:

- For the granite specimens, the pattern, crack coalescences, which are regarded as a function of the bridge angle, and natural crack propagation have been classified.
- Compared with other failure patterns, the anti-wing crack seems to be more common.
- From two 3-D pre-existing surface cracks, a linkage exists between petal cracks inside the specimens and cracking on the surface.
2.1.3.2 Two pre-existing cracks with different inclination angle and bridge angle

Cao et al. (2015) investigated the different geometries of various numbers of flaws under compressive stress. Crack patterns and crack propagation in rock materials were also studied and the DCS-200 loading system was used in this experiment. The loading rate was set up to 50 N/s and the specimens’ external dimensions were 200mm × 150mm × 30mm with the inclination angles of pre-existing cracks being 25°, 45° and 75°. The inclination angles of bridges (β) varied from 25° to 105°. Table 2-2 presents the mechanical parameters, and Figure 2-11 shows the geometry.

Table 2-2 Mechanical parameters of numerical specimen (Cao et al., 2015)

<table>
<thead>
<tr>
<th>Density (g/cm$^3$)</th>
<th>Young's modulus (MPa)</th>
<th>Uniaxial compressive strength (MPa)</th>
<th>Uniaxial tensile strength (MPa)</th>
<th>Poisson's ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.019</td>
<td>0.002272</td>
<td>23.13</td>
<td>2.75</td>
<td>0.2251</td>
</tr>
</tbody>
</table>

Figure 2-11 Geometry of the specimens two pre-existing flaws (Cao et al., 2015)

The specimens’ two pre-existing flaws differ not only in the stress-strain distribution, but also in the crack propagation and failure pattern. Figure 2-12 summarizes the different failure patterns and stress gained. The authors identified two types of cracks initiated from
the flaw tip, cracks, and secondary cracks. This differs from the results found in Yin et al. (2014), where there were only two types of initiated cracks: wing cracks and anti-wing cracks. The reason for this variation is probably the different geometry of the crack and the location of the open flaw.

However, besides the compressive test, Cao and others (2016) performed biaxial, triaxial, tension, shear and cycle loading on the rock-like material. As well as using a high-speed camera, they also conducted a few other monitor methods. They observed the distribution of acoustic emissions, used numerical images and even performed a CT scan technique. Along with the laboratory testing, they also conducted numerical modelling for the propagation.

<table>
<thead>
<tr>
<th>Coalescence mode</th>
<th>Schematic diagram</th>
<th>Crack initiation stress/MPa</th>
<th>Coalescence stress/MPa</th>
<th>Peak strength/MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>T mode</td>
<td><img src="image" alt="Schematic" /></td>
<td>7.24</td>
<td>10.73</td>
<td>28.33</td>
</tr>
<tr>
<td>S mode</td>
<td><img src="image" alt="Schematic" /></td>
<td>4.05</td>
<td>19.07</td>
<td>19.07</td>
</tr>
<tr>
<td>T-S mode</td>
<td><img src="image" alt="Schematic" /></td>
<td>8.21</td>
<td>13.30</td>
<td>22.21</td>
</tr>
</tbody>
</table>

**Figure 2-12 Two pre-opened flaws specimens (Cao et al., 2016)**
2.1.3.3 Two pre-existing cracks with different width and length

Gratchev and others (2016) performed a series of studies on the crack propagation of a rock-like material, concrete. They used concrete with different pre-existing crack sizes under unconfined compression strength (UCS) test and recorded the failure procedure with a high-speed camera.

In their research, Gratchev and others (2016) used the rock bridge to represent the discontinuities of the rock sample. Beside the difference in crack sizes, three different types of concrete were used: commercially Bastion concrete mix (Type A); Type B was made by the mixture of water, cement and coarse basalt aggregates; Type C was made with the same ingredients as Type B, with a difference in the water/cement ratio. All three materials contained cracks with a 50 mm and 70 mm depth and crack widths of 1, 3 or 5 mm (Figure 2-13).

![Figure 2-13 Specimens with pre-existing cracks and (a) a short rock bridge (b) along rock bridge (Gratchev et al., 2016)](image-url)
From Figure 2-14, it can be seen that with the crack width increase, the strength of all three types of concrete decreased, even when the crack depths were different. This shows in the same signs with the increasing the crack depth in the stain-stress curve as well (Figure 2-15). With the increase of the crack width, type A, B and C all indicate the failure stress is decrease. It shows the same results that, with the increase of the pre-existing cracks, the failure stress is decreasing as well.

Figure 2-14 Results from unconfined compression tests on specimens with short and long rock bridges: (a) Type A, (b) Type B, (c) Type C (Gratchev et al., 2016)
Figure 2-15 Strain curves of specimens, Type B (Gratchev et al., 2016)

Beside the UCS result, the point of these researchers’ experiment was to observe the propagation of the cracks until sample failure. Thus, failure propagation and patterns (Figure 2-16 and 2-17) were also discussed.

Figure 2-1617 Crack initiation and propagation of the sample (Gratchev et al., 2016)

Figure 2-18 Failure patterns of different failure sample (Gratchev et al., 2016)
In conclusion, despite involving three types of concrete, the length of the rock bridge was consistently associated with the strength of the rock. The longer the rock bridge is, the higher the strength it has. This was more obvious when the crack width was 1 mm. The width of the crack is a significant factor for the strength of the specimen. The failure for most specimens was caused by shear crack with the coalescence of the pre-existing crack.

2.1.4 Muti pre-existing cracks

2.1.4.1 Non-cross cracks

Cheng et al. (2016) investigated the propagation of pre-existing cracks and the behaviours of coalescence in rock-like material under uniaxial compression with three different arrangements: one pair of flaws was parallel, and the other pre-existing flaws had two different angles with the respective horizontal line (Figure 2-18).

![Figure 2- 1920 Layout for the specimen pre-existing crack (Cheng et al., 2016)](image)
CHAPTER 2

The modelling mixture consists of sand, plaster, limestone and water. The ratio of mass of these is 12:1:2:2, respectively. Table 2-3 shows the basic parametric values of all materials.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit weight</th>
<th>Uniaxial compressive strength</th>
<th>Friction angle</th>
<th>Young's modulus at 50% peak strength</th>
<th>Poisson ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight</td>
<td>16 kN/m³</td>
<td>0.694MPa</td>
<td>0.75</td>
<td>283MPa</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Table 2-3 Mechanical parameters of numerical specimen

In this experiment, the open flaws had two different flaw β angles (135° and 180°) and three crack opening depths (0.5mm, 1mm and 2mm), so tests were conducted on six different numerical samples. 2a and 2b in these tests were fixed at 20mm and 26mm. Tests were performed under the uniaxial compression test system which also included a set of observations using a high-speed camera.

The complete axial stress-strain curves of these rock-like material samples containing three types of pre-existing cracks were also recorded. From Figure 2-19, it is clear that all trends for these six groups of tests are fairly similar to each other. They all went up to a peak with the constant increase of the applied load. During this increase, three stages can be identified. In the first stage, the strain increase mostly corresponds to the closing of the pre-existing cracks. After the initial increase, the growth of the strain approaches linear. When it nearly reaches the peak, it becomes nonlinear, which reveals the occurrence of new cracks. After the peak, the material becomes softly brittle.
Figure 2- 21 Complete strain-stress curve. (a) $\beta = 135^\circ$, (b) $\beta = 180^\circ$ (Cheng et al., 2016)

Figure 2- 22 (a) Relation of depth with peak stress, (b) relation of depth with peak strain (Cheng et al., 2016)

Even from Figure 2- 20, it can be seen that the relationships between the crack depth and the peak stress or strain are quite close. This shows that, with the increase of crack depth, the strain and stress both decreases. This finding is similar to that of Yang and others (2012) who used different types of sandstone.

However, when looking at the failure patterns, there are few differences based on the allocation of the multiple flaws. For those parallel flaws with a larger interdistance, the coalescence between two flaws is normally either a wing crack or anti-wing crack (Zhou et
al., 2014; Yin et al., 2014; Cheng et al., 2016). This type of coalescence always occurs at the tip of the flaw and propagates to the other flaw’s tip. On the other hand, if the distance between two flaws is relatively close, the types of coalescence are quite varied. Instead of wing and anti-wing cracking, quasi-coplanar, oblique secondary and secondary cracks always happen. This phenomenon has been discussed by Park and Bobet (2009, 2010), Yang et al. (2012), Zhou et al. (2014), Cao et al. (2015) and Cheng et al. (2016).

In conclusion, the crack initiation stress decreases with the size of pre-existing open cracking increases and the crack initiation angle decreases as well. Along with that, different types of crack openings occur. With the initiation stress of wing crack and coplanar secondary crack decrease, the crack opening increases. In comparison, for anti-wing cracks, the increase of the stress increases the crack opening. For pre-existing open cracks, the crack propagation normally initiates at the adjacent tips of these pre-existing cracks, no matter whether they are wing cracks or anti-wing cracks. For both wing cracks and anti-wing cracks, neither coplanar secondary and nor oblique secondary cracks appear at the initial stage.

2.1.4.2 Non-cross cracks

It has been found that at low-temperature refrigeration, polymethylmethacrylate (PMMA) is highly transparent, becoming a brittle rock-like material. Zhou et al. (2018) performed a series of tests on PMMA. The specimen’s geometry and the arrangement of pre-existing flaws is shown in the Figure 2-21. For the non-overlapping length c, the samples varied, being -10mm, 0mm and 10mm. The inclination angle $\alpha$ is $30^\circ$, $45^\circ$ and $60^\circ$. 
The experiment also involved using a compressive test machine system and a high-speed camera for image capturing. Figures 2-22 and 2-23 show experimental results with different inclination angles and non-overlapping lengths.

Figure 2- 23 Specimens geometry (a) front view (b) vertical view (c) cross-section schematic (Zhou et al., 2018)

Figure 2- 24 Stress versus (a) inclination angle and (b) non-overlapping length (Zhou et al., 2018)
From the results of the inclination angle and crack initiation stress curve and the results for the three different non-overlapping lengths, a similar trend can be observed: the lowest stress appears as the first crack initiation stress at $45^0$, and for the same angle, with the non-overlapping length increasing, the crack initiation stress increases as well.

Zhou and others (2018) also investigated crack propagation and failure patterns for all conditions. They identified six stages in the process of cracking:

- Crack initiation: In this stage, wing cracking is commonly expected for all the specimens. After these wing cracks happened, oblique secondary cracks occurred.
- Cracking propagation: With the continued applied load on the sample, the wing cracks normally propagated towards the loading direction. In this stage, the propagating cracks always started to coalescence or wrap.
• **Crack coalescence:** The coalescence of wing cracks always takes place before crack wrapping. Petal cracks occurred after anti-wing cracks appeared, which was also later than the wing cracks’ occurrence. The ending with the petal cracks coalescence finished.

• **Substantial crack wrapping:** The failure pattern indicates that the petal cracks normally propagated or wrapped from wing cracks. Compared with other two types of cracks mentioned above, petal cracks are more symmetrical.

• **Mixed crack wrapping and coalescence:** At this stage, the petal cracks wrap and coalescence. Cracking propagation of the specimen is nearly finished.

• **Failure:** This is the end of the compressive process where the specimen fails.

The difference in petal cracks remains to be discussed. In 2003, Dyskin and others defined a kind of wrapping behaviour called substantial wrapping. It is quite rare to record the wrapping procedure because normally the samples are made of concrete or some other non-transparent material. Dyskin measured both open mode and closed mode wrapping.

In conclusion, the compressive behaviour for a specimen with two open flaws shows it has an elastic deformation which is quite common and a brittle failure. However, for specimens with a small non-overlapping length \((c=0 \text{ or } -10 \text{ mm})\), the effect of the interaction of two open flaws is significant.

In terms of cracking typology, three observations can be made. There are four modes for initiation cracks: wing cracks, anti-wing cracks, oblique cracks and petal cracks. Five different types of crack coalescence exist: coalescence of wing cracks and pre-opened cracks, coalescence of wing cracks, coalescence of wing cracks and petal cracks, coalescence of petal cracks, and coalescence of anti-wing cracks with petal cracks. There are also two types of petal cracks: opened and closed.
2.1.5 Summary of crack propagation experiments

From the research outlined above, the most widely used material is concrete or PMMA. However, rock is not always as uniform as these two types of material. Therefore, part of the present study involves focusing on the different behaviours of natural rocks. Table 2-4 summarises the different materials and the methods used for some of the main test studies identified in the literature.

Table 2-4 Summary of tests

<table>
<thead>
<tr>
<th>Year</th>
<th>Material</th>
<th>Type of loading</th>
<th>Type of recording</th>
</tr>
</thead>
<tbody>
<tr>
<td>1993</td>
<td>Concrete</td>
<td>Shear force</td>
<td>Strain sensor</td>
</tr>
<tr>
<td>1998</td>
<td>Brittle rock</td>
<td>Uniaxial testing</td>
<td></td>
</tr>
<tr>
<td>2004</td>
<td>Brittle rock</td>
<td>Uniaxial compressive stress</td>
<td>Strain sensor</td>
</tr>
<tr>
<td>2005</td>
<td>Marble</td>
<td>Uniaxial compressive stress</td>
<td></td>
</tr>
<tr>
<td>2006</td>
<td>Sandstone</td>
<td>Uniaxial hydraulic load</td>
<td>Real-time holographic interference method</td>
</tr>
<tr>
<td>2009</td>
<td>Gypsum</td>
<td>Uniaxial compressive stress</td>
<td>Camera and strain sensor</td>
</tr>
<tr>
<td>2013</td>
<td>Concrete</td>
<td>Uniaxial compression and uniaxial tension</td>
<td>Lab experiments with numerical model</td>
</tr>
<tr>
<td>2014</td>
<td>Numerical model</td>
<td>Uniaxial compressive stress</td>
<td></td>
</tr>
</tbody>
</table>
2.1.6 Crack failure propagation summary

The research has investigated the failure of the rock-like material and the analysed the failure pattern of the material, therefor the main conclusion has been summarized as follow:

- Basically, all the cracks initiation from the original opened flaws, and normally all the cracks are wing cracks.
- Different pre-existing cracks induced same initially cracks lead to different following up cracks: secondary cracks, and anti-wing cracks.
- Some pre-existing cracks in the middle of the specimens under compressive load can cause another failure pattern at the end. At the stage of coalescence, the coalescence of the cracks would wrap and cause a petal crack.
2.2  Studies of rock joint with and without infill under shear stress

Infilled material also appears in between a rock joint. An infilled rock joint is one of the common structures in the field. This section is an introduction to the shear strength of rock joint with and without infill material.

2.2.1 Basic study of shear strength

Barton and Choubey (1977) first defined the roughness of rock joint as a coefficient which they termed the joint rough coefficient (JRC). The JRC value is depicted as a waved line by using Barton’s comb. They also visually profiled roughness corresponding to JRC ranges. By comparing different typical roughness profiles, the JRC value can be determined. Based on the JRC values, Barton and Choubey (1977) presented the following mathematical equation to calculate the shear strength:

\[ \tau = \sigma_n \tan (\varphi_r + JRC \log \left( \frac{JCS}{\sigma_n} \right)) \]

Where:

\( \tau \): shear strength of jointed rock (MPa).
\( \sigma_n \): normal stress what applying on the jointed rock (MPa).
\( \varphi_r \): residual friction angle of rock (°)

JRC: joint roughness coefficient
JCS: unconfined compression stress of rock (MPa).

By using this equation, the shear strength of clean jointed rock can be accurately calculated.

Kim and others (2013) and (2016) endorsed the JRC value, affirming its vital importance when determining the shear strength of rock mass. As a preliminary test,
Barton’s comb has been widely used to measure the JRC value in the field, however, it is not the most efficient method in terms of both labour and time. Therefore, Kim and others (2013) performed a series of tests by using a tilt test with different saw-tooth models to measure the JRC value, the accuracy of which is far greater than that measured by Barton’s comb. Because of the difference of the actual JRC values and the values as measured by the comb, Eqs. 2.6 and 2.7 are used to back-calculate the JRC value.

\[
JRC = \frac{\alpha - \phi_r}{\log_{10} \left( \frac{JCS}{\sigma_n} \right)} 
\]

(2.6)

\[
\phi_r = (\phi_b - 20) + 20 \times \left( \frac{r}{R} \right) 
\]

(2.7)

Where:

\(\alpha\): orientation of the joint (°)

\(\phi_r\): residual friction angle (°)

\(\phi_b\): basic friction angle (°)

\(r\): the rebound number on weathered joint surface from the Schmidt hammer test

\(R\): the rebound number on unweathered joint surface from the Schmidt hammer test.

For the basic friction angle, Kim and others (2013) used a rough surface joint against an unweathered smooth surface. Patton (1966), Krsmanovic (1967) and Coulson (1972) found that, for sandstone, the basic friction angle is about 30° to 35°.

Table 2-5 Mean R for sandstone

<table>
<thead>
<tr>
<th>Study</th>
<th>Mean R value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gokceoglu &amp; Aksoy (2000)</td>
<td>18.3-33.6</td>
</tr>
<tr>
<td>Goudie, Migoń, Allison, &amp; Rosser (2002)</td>
<td>41-44.7</td>
</tr>
<tr>
<td>Yaşar &amp; Erdoğan (2004)</td>
<td>44.5</td>
</tr>
<tr>
<td>Saptono et al. (2013)</td>
<td>10-28</td>
</tr>
</tbody>
</table>
As for both \( r \) and \( R \), which represent the rebound values of weathered and unweathered joints, respectively, these can be measured by the travel distance of a piston after it rebounds. The fresh joint rebound value \( R \) is such that, the harder the rock is, the higher the \( R \) value. Table 2-5 shows some of the mean \( R \) values for sandstone.

Singh et al. (1983), O’Rourke (1989), Katz et al. (2000) and Saptono et al. (2013) affirmed the correlation between the hardness character of rock (UCS) and the \( R \) values (see Table 2-6). rock.

### Table 2-6 Correlation between the \( R \) value and UCS

<table>
<thead>
<tr>
<th></th>
<th>Correlation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Singh et al. (1983)</td>
<td>( \text{UCS} = 2R )</td>
</tr>
<tr>
<td>O’Rourke (1989)</td>
<td>( \text{UCS} = 702R - 1104 )</td>
</tr>
<tr>
<td>Katz et al. (2000)</td>
<td>( \text{UCS} = 2.208e^{0.067R} )</td>
</tr>
<tr>
<td>Saptono et al. (2013)</td>
<td>( \text{UCS} = 0.308R^{1.327} )</td>
</tr>
</tbody>
</table>

Even though it is convenient to link the \( R \) values to the UCS, there are also limitations, as presented by Goudie (2006). The Schmidt hammer test is extremely sensitive to the water content in soft rocks, also the hammer can easily interfere with the discontinuities, which potentially damages the

#### 2.2.2 Basic study of shear strength with infill

When there is infill in between the jointed rock, the shear strength of the structure is changed. Based on the difference in loading conditions, there are two distinct types of the direct shear test: constant normal load (CNL) and constant normal stiffness (CNS).
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Constant normal load means that during the shear test, the normal load of the sample remains constant. Under the CNL condition, the applied normal stress is more constant which makes the shear plane interface more realistic.

Previous research indicates that the most widely used type of joint model is the saw-tooth model. As shown in Fig.1-3, the difference between the saw-tooth model and a naturally occurring rough surface is quite obvious.

Similarly, to the unconfined compression strength study which presented in Chapter 2.1, the material used in the joint model is more likely to be artificial instead of a core sample taken from natural rock.

2.2.2.1 Shear test under constant normal load

Lama (1978) performed a series of tests on rock joint with infill, based on which he proposed a theory to calculate the shear strength with infill material:

\[ T_p = 7.25 + 0.46 \times \sigma_n - 0.3 \times \ln(f) \times \sigma_n^{0.745} \]  

Where:

- \( T_p \): shear strength (MPa)
- \( \sigma_n \): normal strength (MPa)
- \( f \): thickness of infill material.

The limitation of this equation is that though it takes the thickness of infill into consideration, the roughness of the joint is not accounted for. According to previous research, the roughness of the joint is vitally important when calculating the shear strength.

Papaliangas and others (1993) and Jahanian and Sadaghiani (2015), made a modification of calculations of the shear strength related with the t/a, which has been proposed as:

\[ \mu = \mu_{min} + (\mu_{max} - \mu_{min})^n \]  

Where:

- \( \mu \): coefficient of friction
- \( \mu_{min} \): minimum coefficient of friction
- \( \mu_{max} \): maximum coefficient of friction
- \( n \): a parameter to control the variation of \( \mu \) from \( \mu_{min} \) to \( \mu_{max} \).
$\mu_{\text{min}} = \frac{T_{\text{min}}}{\sigma} \times 100 \quad 2.10$

$\mu_{\text{max}} = \frac{T_{\text{max}}}{\sigma} \times 100 \quad 2.11$

$n = \left(1 - \frac{1}{c} \times \frac{t}{a}\right)^m \quad 2.12$

Where:

$\mu$: shear strength (MPa).

C and m: experimentally derived constant,

$T_{\text{max}}$: maximum shear strength of unfilled joints (MPa).

$T_{\text{min}}$: potential minimum shear strength (MPa).

Jahanian and Sadaghiani (2015) performed a series of direct shear box tests using a saw-tooth model (see Figure 1-3). The material for the model was gypsum plaster.

In this experiment two different types of joints were involved: Type 1 had a 30° saw-tooth angle and Type 2 had a 45° saw-tooth angle (see Figure 2-24). The normal stresses applied on the joints were 30, 60, and 120 kPa for both types. The t/a ratio for Type 1 was 0.5, 1 and 2 and for Type 2 joint it was 0.3, 0.6, 1.2. The infill material of the joint was sandy clay which is one of the common infill materials in actual rock discontinuities.

Figure 2-26 Cross-section of joint

Figure 2-25 shows the shear strength envelope for both joints. The friction angle of the infilled joint has a greater deduction in the strength with a sharper asperity angle.
However, the influence of the sandy clay infill for joints with different asperity angles is not significant.

Figure 2-27 Shear strength envelope: (a) type 1 joint, (b) type 2 joint (Jahanian and Sadaghiani, 2015)

Jahanian and Sadaghiani (2015) also verified that for the infill under a low normal stress can contribute more to the strength of the rock joint which compared with a higher normal stress, the asperity is. A joint with a small asperity angle is less affected by the increases of the normal stress. Their study also covered the investigation of the critical t/a value. For both Type 1 and Type 2 joints, the critical t/a ratios (t/a)cr were both about 1.2.

Naghadehi (2015) performed similar tests, studying the shear behaviour of sandstone with infilled material. The sandstone was collected from Iran and not sheared naturally before sampling. Samples were transported to the laboratory and manually cut into saw-tooth shapes. The used sandstone had a 25-30 MPa unconfined compressive strength and the Young’s modulus was equal to 4.8-5.5 MPa. Three types of infill were used in this study: sand, clay and sandy clay. The t/a ratios ranged from 0 to 1.6 with a 0.2 increment, under 0.25, 0.5, 0.75 and 1 MPa constant normal loads.

From the test results, it is clear that no matter which type of infill, the shear strength of an infilled joint is less than the one from a clean joint. Also, with the increase of infill thickness, the shear strength of the infilled joint decreases. When the t/a ratio is over 1.4, the
shear strength is a close to that of the infill material instead of that of the joint. This indicates that, with this infill or a thicker infill, the shear will not take place in between the joint surface and the infill material. Instead, it mainly proceeds inside the infill itself. Naghadehi (2015) also presented a final equation for the prediction of the shear strength with infill:

\[
\frac{\tau_p}{\sigma_n} = \frac{\tau_0}{\sigma_{n0}} - \frac{c(t/a)}{\sigma_n[(t/a)^2 + k_1(t/a) + k_2]}
\]

2.13

Where:

\( \tau_p \): shear strength of infilled joint (MPa).

\( \tau_0 \): shear strength of clean joint under normal load (MPa).

\( \sigma_n \): applied normal strength (MPa).

\( t \): infill thickness (mm).

\( a \): average roughness of joint surface (mm).

\( k_1 \): empirical constants.

\( k_2 \): empirical constants.

\( c \): empirical constants.

In Eq. 2.12, \( k_1 \), \( k_2 \) and \( c \) vary due to the different normal loads and the type of infill materials. With this prediction method, Naghadehi (2015) used RMSE (= 0.2501) which is a root mean square error to evaluate the accuracy of this equation. Compared with previous results (Goodman, 1970; Lama and Butukuri, 1978; Phien-Wej et al., 1990; De Toledo et al., 1993) are from 0.1571 to 0.3636, this method is relatively accurate.

**Table 2-7 Summary of the RMSE value**

<table>
<thead>
<tr>
<th></th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Goodman (1970)</td>
<td>0.1571</td>
</tr>
<tr>
<td>Lama and Butukuri (1978)</td>
<td>0.2120</td>
</tr>
</tbody>
</table>
Lu and others (2017) investigated the shear strength of infilled rock joint with acoustic emission (AE) and photography monitoring techniques. As with previous research, the main parameters of this study were the asperity height (7.5mm) and inclination angle (45°). The sample they used was sandstone with a 79.3 MPa UCS strength. The sample was cut into saw-tooth shape. The infill used was concrete with a water-cement ratio of 1:2. The asperity height over thickness ratios were 0, 0.05, 0.1, 0.25, 0.5, 0.7, 1, 1.5 and 2.

Based on their test results, Lu and others (2017) summarized the shear stress vs shear displacement curve as consisting of three different types:

- **Type 1** is controlled by “joint wall-joint wall” interaction. This means that the shear mostly happens in the unfilled joints or with a low t/a ratio ($0 \leq t/a \leq 0.05$) joint. A sliding phase was observed before a strain softening phase after the peak shear stress was reached.
- **Type 2** is controlled by “joint wall-hardened cement-grout-joint wall”. This indicates that the shear can be monitored at an intermediate filling ratio ($0.05 \leq t/a \leq 1$). An abrupt softening phase was noted after the maximum shear stress value was reached.
- **Type 3** is controlled by “hardened cement grout”. This is when the shear happens in a joint with a high t/a ratio ($t/a \geq 1$). In the stress-displacement curve, there is no softening phase, which means that after the shear reaches the maximum, it remains at almost same level as this peak value.

The shear strength of the cement grout-filled joint was analysed by a function of the filling ratio based on the following empirical formula:
\[ \tau_p = c_1 \exp(-c_2 \frac{t}{a}) - c_3 \exp(-c_4 \frac{t}{a}) + c_5 \]  

In this equation, \( c_1, c_2, c_3, c_4, c_5 \) are positive constants. It contributes to the shear strength of the joint. The first part represents the peak shear strength contributed by the joint wall-joint wall intersections. The second part reflects the load-bearing capacity of the hardened cement grout. The last part reflects the interface cementing effect.

In conclusion, the strength of the infilled material as cement joint is also highly affected by the ratio of thickness and the asperity height. According to AE monitoring, no matter whether the joint is clean or filled with cement, there is a quiescent period of AE before it reaches peak shear stress. This also indicates that every evident decrease of the stress-displacement curve is accompanied by a surge of AE.

Zhao and others (2020) performed a series of direct shear tests on joints made by rock-like material (cement). The mechanical properties of this material are shown in Table 2-7.

<table>
<thead>
<tr>
<th>Property of rock like material.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus (GPa)</td>
</tr>
<tr>
<td>2.45</td>
</tr>
</tbody>
</table>

This rock-like material was made into a 100 mm × 65 mm × 65 mm block. Instead of saw-tooth, this study used a premade rough surface with different JRC values according to Barton’s profile. The infill used in this study were poorly graded sand (SP) and lean clay (CL). The moisture contents of the infill were 5.91% for the sand and 20% for the clay.
Zhao and others (2020) summarized their shear stress (SS) envelopes (Figure 2-26) based on their test results according to the Coulomb criterion. They also presented the normalized peak shear strength ($\tau_p/\sigma_n$) vs the t/a ratio (Figure 2-27).

Like Lu and others (2017), Zhao and others (2020) summarized their shear displacement vs shear stress curve into two different types. The first is the hump type (H type). This type has a linear relationship at the beginning of the displacement vs stress curve. After reaching the peak value, there is a softening phase of the strain, or the shear strength becomes nearly stable. The H type shear stress curve was mostly observed in samples with the least filled joint or high JRC values. The second type is the gentle type (G type) which also has linear deformation in the initial stage. However, the difference between it and the H type is that there is no softening phase for the G type. The shear strength of this type of sample almost remains stable or increases slightly after the peak value is reached.

In addition to the shear displacement vs shear stress curve, Zhao and others (2020) also characterized the normal displacement vs shear displacement into three different types: pure dilatation (D type), pure compression (C type) and compression-dilation (C-D type). The D type represents the shear displacement, and the normal displacement curve shows a linear variation. The C type stands for the deformation of the normal direction. It is not dilation, which means the displacement is compression. For the C-D type, there is a compression stage in the beginning and after that, the normal deformation changes into dilation.
Figure 2-28 SS envelopes based on Coulomb criterion: (a) sand infilled joints (b) clay infilled joint (Zhao et al., 2020)

Figure 2-29 Normalized peak shear strength ($\tau_p/\sigma_n$) vs the t/a ratio for clay (Zhao et al., 2020)
The following two equations (Eqs. 2.13 and 2.15) summarize the test results (Zhao et al., 2020). By using these two equations and the shear strength of infill material under different normal stress, the boundary of the interfering zone can be determined. The interfering zone represents the critical values for the thickness of infill with different asperity heights:

\[
\frac{\tau_p}{\sigma_n} = 0.533 + 1.3e^{-2.7\frac{t}{a}} \quad 2.15
\]

\[
\frac{\tau_p}{\sigma_n} = 0.512 + 2.88e^{-4.3\frac{t}{a}} \quad 2.16
\]

Additionally, as Xu et al. (2013) devised, the relationship between the friction angle of the infield joint and \(\varphi_{ij}\) and \(t/a\) is:

\[
\varphi_{ij} = \varphi_p \exp\left(-\beta \frac{t}{a}\right) \quad 2.17
\]

Where:

\(\beta\): a decay coefficient.

Zhao and others (2020) analysed the previous JRC values vs the \(\beta\tan\varphi_{infill}\) distribution from the experimental data of Papaliangas et al. (1990, 1993), Naghadehi (2015), Jahanian and Sadaghiani (2015) and Zare (2015). The upper and lower boundaries of the distribution have been summarized as Eqs. 2.18 and 2.19. Eq. 2.20 represents the average distribution investigated from the previous research (Jahanian and Sadaghiani 2015).

\[
\beta = \frac{0.022JRC + 0.055}{\tan \varphi_{infill}} \quad (Upper \ boundary) \quad 2.18
\]

\[
\beta = \frac{0.015JRC + 0.017}{\tan \varphi_{infill}} \quad (Lower \ boundary) \quad 2.19
\]

\[
\beta = \frac{0.017JRC + 0.029}{\tan \varphi_{infill}} \quad 2.20
\]
According to Zhao and others (2020), the infills play important roles in the shear process. Therefore, the calculation for shear stress of the infilled joint is Eq. 2.21 when the t/a is less than \((t/a)_{cr}\) and Eq. 2.22 when the t/a greater than \((t/a)_{cr}\):

\[
\tau_p = \sigma_n \times \tan \left\{ \left[ JRC \times \log \left( \frac{JCS}{\sigma_n} \right) + \varphi_p \right] \times \exp \left( -\beta \times \frac{t}{a} \right) \right\} + c_{\text{infill}} \quad 2.21
\]

\[
\tau_p = \sigma_n \times \tan \varphi_{\text{infill}} + c_{\text{infill}} \quad 2.22
\]

In conclusion, Zhao and others (2020) pointed out that the shear strength model based on the Barton-Choubey theory for infilled joint requires the relation between the friction angle and the t/a ratio for the shear strength to be predicted.

2.2.2.2 Shear test under constant normal stiffness

This final section comprises a description of previous research about direct shear tests for infilled joint under constant normal load (CNL). A further method is introduced to perform the direct shear test. CNL means that, during the whole shear process, the normal load applying on top of the rock joint remains constant. Constant normal stiffness (CNS) refers to the condition where, during the shear process, the stiffness of the rock joint structure remains constant. Compared with CNL, CNS is much more complicated. Figure 2-28 shows one of the common basic structure frames of the constant normal stiffness test.

![Two-dimensional model for CNS test.](image)
In Figure 2-28, $k_n$ is the normal stiffness and $\delta_n$ is the joint dilation. This testing method is much more suitable for underground excavations and closer to the natural filled condition (Johnstone and Lam, 1989; Ohnishi and Dharmaratne, 1990; Skinas et al., 1990; Haberfield and Johnstone, 1994; Indraratna et al., 1999).

Indraratna and others (1999) performed both CNS and CNL direct shear tests on a saw-tooth model made of plaster. There were two types of the joint inclinations, Type 1 was $9.5^\circ$ and Type 2 was $18.5^\circ$ and the $k_n$ was $8.5\text{kN/mm}$. The initial normal stresses were 0.3, 0.56 and 1.1 MPa.

As Duncan and Chang (1970) defined, the normalized strength drop (NSD) is related to the $t/a$ ratio:

$$NSD = \frac{t/a}{a(t/a) + \beta}$$

Indraratna was of the view that, with regard to various field applications, especially in mining excavations, testing the shear behaviour of rock joints with CNS is more accurate than with CNL. Regarding changes in normal stress and joint dilation during shearing, test results obtained under CNS differ from those obtained under CNL.

Regardless of whether the tests are performed under CNS or CNL, a critical factor during shearing is the ratio of the infill thickness to the asperity height ($t/a$). According to laboratory tests, the shear plane hits the “crown” of the toothed asperity for $t/a$ ratios close to one and goes through the asperity and infill for $t/a$ ratios less than unity. Depending on the original normal stress, only for $t/a$ ratios greater than a critical value which various depending on the different type of the rock does the shear plane pass through the infill. In other words, the shear behaviour is primarily controlled by the infill alone until a certain $t/a$ ratio is exceeded. When compared to previous researchers’ proposals based on CNL testing, the essential $t/a$ ratio obtained for CNS tests is appreciably smaller.
CHAPTER 2

Patton (1966) described the shear strength of a rough joint with regular asperities as:

\[
\tau = \sigma_n \tan(\varphi_b + i)
\]

Where:

\(i\): angle of asperity.

With this equation as a foundation, Indraratna and others (1999) presented a prediction of infilled shear strength:

\[
\tau_{(h,\text{CNS})} = (\sigma_{n0} + \Delta\sigma_{nh}) \left[ \frac{\tan(\varphi_b + \tan(i))}{1 - \tan(\varphi_b)\tan(i_h)} \right]
\]

Where:

\(\tau_{(h,\text{CNS})}\): joint shear stress at a horizontal displacement of h.

\(\sigma_{n0}\): corresponding normal stress

\(i_h\): dilation curve at a horizontal displacement h.

Even though this model can predict the shear strength of a jointed infilled rock relatively closely, the model is rather dependent on the regular shape of the asperity. In the field, rock joints are rougher and more irregular. In addition, sometimes particle breaks occur during the shear process, which will change the initial condition.

Indraratna and others (2005) proposed a limit to the asperity height and thickness ratio. They believed that the t/a ratio will change with variations of the asperity angle. They assumed that the normalized shear strength can be presented as two algebraic functions, A and B (Eqs. 2.27 to 2.29), in which the t/a ratio is less than the critical ratio:

\[
\left( \frac{\tau_p}{\sigma_n} \right)_{OC,\text{CNS}} = A + B
\]

\[
A = \tan(\varphi_b + i) \times (1 - k)^{\alpha}
\]

\[
B = \tan\varphi_{fill} \times \left( \frac{2}{1 + 1/k} \right)^{\beta}
\]

Where:

\(\alpha, \beta\): empirical constant.
k: ratio of t/a over (t/a)_{cr}.

i: initial asperity angle.

This algebraic function can be only used when the t/a ratio is less than the critical t/a ratio, which means $k$ is less than 1. When $k$ is less than 1, Indraratna and others (2005) suggested this implied existence of an interfering zone.

![Figure 2-31 Shear strength model for infilled joints (Indraratna et al., 2005)](image)

As can be seen in Figure 2-29, two different zones can be discerned: an interfering and a non-interfering zone. In the interfering zone, the normalized shear strength is equal to the summation of algebraic functions A and B. However, when the t/a is greater than the (t/a)_{cr}, the normalized shear strength is equal to the tan$\phi_{infill}$.

Indraratna and others (2005) took a further step in their investigations by presenting a list (Table 2-8) of the empirical constants for different types of joints and materials.
Table 2-9 Summary of empirical constants

<table>
<thead>
<tr>
<th>Joint type</th>
<th>Type of infill</th>
<th>(t/a)\text{cr}</th>
<th>(\alpha)</th>
<th>(\beta)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1</td>
<td>Graphite((\varphi_{\text{infill}} = 21^\circ))</td>
<td>1.2</td>
<td>1.7</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>Bentonite((\varphi_{\text{infill}} = 25^\circ))</td>
<td>1.5</td>
<td>1.2</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>Clayey sand((\varphi_{\text{infill}} = 30^\circ))</td>
<td>1.4</td>
<td>1.1</td>
<td>2.5</td>
</tr>
<tr>
<td>Type 2</td>
<td>Graphite((\varphi_{\text{infill}} = 21^\circ))</td>
<td>1.4</td>
<td>1.5</td>
<td>2.2</td>
</tr>
<tr>
<td></td>
<td>Bentonite((\varphi_{\text{infill}} = 25^\circ))</td>
<td>1.8</td>
<td>1.1</td>
<td>3.1</td>
</tr>
<tr>
<td></td>
<td>Clayey sand((\varphi_{\text{infill}} = 30^\circ))</td>
<td>1.6</td>
<td>1.1</td>
<td>4.4</td>
</tr>
</tbody>
</table>

These researchers suggested that the findings of the experiment show that the shear strength can be reduced from the maximum value associated with clean, rough joints by adjusting the ratio of infill thickness to asperity height \((t/a)\). They identified that the influence of the asperities was muted for joints filled with graphite and bentonite when the infill thickness exceeded a certain \(t/a\) ratio, and the infill had the greatest influence on the shear behaviour. For thin clay infills, the shear is mainly dominated by the rock-to-rock surface (Oliveira et al., 2009), and the dilation and suction do not increase with the increased confining pressure (Indraratna and Mylvaganam, 2005).

Based on Eq. 2.27, Indraratna and others (2008) consolidated their findings into this model (Eqs. 2.30 and 2.31):

\[
A = \tan(\varphi_b + i) \times (1 - k_{OC,n})^{a_n} \tag{2.30}
\]

\[
B = \tan \varphi_{\text{fill}} \times OCR^\alpha \times \left(\frac{2}{1 + 1/k_{OC,n}}\right)^{b_n} \tag{2.31}
\]

Similarly to previous research (Indraratna et al., 2005), the model quantified the shear strength for different infill thicknesses over different idealised saw-tooth joints. With the increase of the infill thickness, the shear strength of the infilled joint decreases under
different over-consolidation ratio (OCR) conditions. These researchers indicated that with a lower t/a, the dilation of the infill and the joint reduced the pore water pressure from the infill material. No matter what OCR condition, the pore water pressure will build up and with a relatively high t/s ratio, the influence of the asperity will be suppressed by this excess pore water pressure. In this situation, the reaction of the infill resembles that of an over-consolidated clay.

Indraratna and others (2010) went on to present a new model which can describe the joint shear criterion, taking into account some factors that could influence the shear behaviour. These factors are the fiction angle of the material, the basic friction angle of the rock joint, the degradation of the asperities and the infill thickness. This indicates that the overall stress-strain behaviour of the joint can be described by the rock joint and rock infill components. However, this model still contains several empirical constants and more experimental research using different infill and joint materials is required to prove it ubiquitous applicability.

In 2014 Indraratna and others modified these equations to take into account the degree of saturation of the infilled material. The algebraic function $(\tau_p/\sigma_n)$ remains the same as that proposed by Indraratna and others in 2005. Different calculations for A and B were devised, as shown in Eqs. 2.32 and 2.33:

\[
A = [\sigma_n \times \tan (\varphi_b + JRC \times \log (\frac{JCS}{\sigma_n})) \times (1 - k_s)\alpha] \\
B = C_t + [ (\sigma_n - u_a) \times \tan \varphi'] \times (\frac{2}{1 + \frac{1}{k_s}})^\beta
\]

Where:

$k_s$: t/a ratio over the critical t/a ratio.

$U_a$: pore air pressure which is normally considered as the normal air pressure.

The shear strength model for infilled joint can be estimated by Eqs. 2.34 and 2.35:
\[
\tau = [\sigma_n \times \tan (\varphi_b + JRC \times \log \left(\frac{JCS}{\sigma_n}\right) \times (1 - k_s)^{\alpha} + C_t + \left(\sigma_n - u_a\right) \times \tan \varphi'] \times \\
\left(\frac{2}{1 + k_s}\right)^{\beta} (k_s < 1)
\]

\[
\tau = c_t + (\sigma_n - u_a)\tan\varphi' (k_s > 1)
\]

2.2.3 Summary of the main literature of infilled jointed rock shear strength

The Table 2-9 shows some of the main research of infilled jointed shear strength.

**Table 2-10 Summary of previous research into infilled jointed rock shear strength, with t/a ratio and infill material**

<table>
<thead>
<tr>
<th></th>
<th>Type of model</th>
<th>Material of model</th>
<th>Thickness over asperity (t/a)</th>
<th>Infill material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Goodman (1970)</td>
<td></td>
<td></td>
<td>1.25</td>
<td>Crushed mica</td>
</tr>
<tr>
<td>Tulinevand Melokov (1971)</td>
<td></td>
<td></td>
<td>5 and 6mm</td>
<td>Sandy clay</td>
</tr>
<tr>
<td>Papaliangas et al. (1993)</td>
<td>Rough surface</td>
<td>Plaster</td>
<td>1, 1.5, 1.14</td>
<td>Kaolin, marble dust, dry cohesion soil</td>
</tr>
<tr>
<td>Indraratna et al. (2005)</td>
<td>Saw-tooth</td>
<td>Gypsum plaster</td>
<td>0.6, 1.2, 1.8, 3.6</td>
<td>Graphite and bentonite</td>
</tr>
<tr>
<td>Indraratna and Mylvaganam (2005)</td>
<td>Saw-tooth</td>
<td>Gypsum plaster</td>
<td>0, 1, 2, 3.5</td>
<td>Silty clay</td>
</tr>
<tr>
<td>Indraratna et al. (2008)</td>
<td>Saw-tooth</td>
<td>Gypsum plaster</td>
<td>0.5, 1, 1.5, 2, 3.5</td>
<td>Natural silty clay</td>
</tr>
<tr>
<td>Oliveira et al. (2009)</td>
<td>Saw-tooth</td>
<td>Gypsum plaster</td>
<td>0.5, 1, 1.5, 2</td>
<td>Sandy clay</td>
</tr>
<tr>
<td>Authors</td>
<td>Model</td>
<td>Material</td>
<td>Infill Thickness</td>
<td>Notes</td>
</tr>
<tr>
<td>-------------------------</td>
<td>------------------------</td>
<td>---------------------------</td>
<td>--------------------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td>Indraratna et al. (2014)</td>
<td>Saw-tooth model</td>
<td>Gypsum plaster</td>
<td>0.26, 0.51, 1.53, 2.05</td>
<td>Silty clay</td>
</tr>
<tr>
<td>Naghadehi (2015)</td>
<td>Saw-tooth model</td>
<td>Sandstone</td>
<td>0, 0.2, 0.4, 0.6, 0.8, 1, 1.2, 1.4, 1.6</td>
<td>Sand, clay, sandy clay</td>
</tr>
<tr>
<td>Jahanian and Sadaghiani (2015)</td>
<td>Saw-tooth model</td>
<td>Gypsum plaster</td>
<td>0.3, 0.5, 0.6, 1, 1.2, 2</td>
<td>Sandy clay</td>
</tr>
<tr>
<td>Liu et al. (2017)</td>
<td>Saw-tooth model</td>
<td>Sandstone</td>
<td>0, 0.05, 0.1, 0.25, 0.25, 0.5, 0.75, 1, 1.5, 2</td>
<td>Cement</td>
</tr>
<tr>
<td>Zhao et al. (2020)</td>
<td>Rough surface</td>
<td>Cement</td>
<td>0.614, 0.725, 1.03, 1.89</td>
<td>Sand and clay</td>
</tr>
<tr>
<td>Li et al. (2022)</td>
<td>Saw-tooth model</td>
<td>Sandstone, mudstone and cement</td>
<td>1 mm(infill)</td>
<td>Cement</td>
</tr>
</tbody>
</table>

2.3 Concluding remarks

In this chapter, the related literature has been reviewed. This includes a review of rock behaviour under compression with different discontinuities and shear behaviour with different infill thicknesses:

- There are several different numbers of pre-existing cracks have been studied, as also the different locations of multiple pre-existing cracks. The numbers of pre-existing cracks are from 1 to 3 and the location of these cracks varied from parallel to with different angles. This part of the study has been well conducted to investigate the failure mechanism of rock-like materials. The use of methods includes different experimental procedures and different monitoring methods such as a photographic method or a holographic interference method.
• The previous studies of rock joints covered the testing from different materials of the model, different types of model and different infill thicknesses of the model. The main types of rock joints are rough surface or saw-tooth model. The majority of studies used plaster or gypsum, as rock-like materials, to study the shear behaviour of jointed rock. It also mentioned the different numerical calculations to identify the shear strength of the infilled joint.

Several different shear behaviour models to predict the shear strength with infill have also been outlined, with supporting mathematical content. Yet, there still remain some issues which have not been addressed well in prior research and these warrant further investigation:

• For both unconfined and shear behaviour of infilled rock sample, only limited research has been conducted on natural rock sample with natural infills.

• Insufficient studies exist using rock samples from Southeast Queensland.

In this chapter, the research gaps of the current study have been identified. This has prompted the research presented in this thesis.

The following chapter provides the experimental program to study both unconfined and shear behaviour with different natural rocks with different parameters, such as different depths of discontinuities, different infill materials or different infill thicknesses. This information aids in our understanding of the failure behaviour of the rock from Southeast Queensland.
CHAPTER 3: LABORATORY EXPERIMENTS

3.1 Testing plan

The experimental testing for the present research comprises three different stages: the first concerned the sample collection and preparation. The samples were collected from a local Gold Coast geotechnical company. Density and porosity tests for the sample were conducted after remodeling, and an X-ray diffraction (XRD) test for the mineral composition of the rock sample was performed. In this phase, sample casting for the direct shear test was also done.

The second stage involved conducting the UCS tests with a high-speed camera for all samples. During this procedure, a certain moisture content for the sand and clay samples was determined as the crack infill. The direct shear box test was performed at this stage as well.

The last stage constituted observing the crack propagation and inner viewing of the cracks with a microscope.

3.2 Soil testing

In this study, some soil (sand and clay) has been used. To determine the property of soil, several tests were conducted. Their purposes and test specifications are outlined below:

- Soil distribution and sieve analysis: to determine the practical size of used soil following AS 1289.3.6.1-2009.
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- Soil classification test, liquid limit and plastic limit: to classify the type of the clay sample using AS 1289.3.1.1-2009 and AS 1289.3.2.1-2009.
- Soil density test: to determine the dry density and the moisture content by AS 1289.5.1.1:2017.
- Soil shear strength testing: to test soil shear strength and consolidation via AS 1289.6.2.2:2020.

3.3 Rock quality designation test

Figure 3-1 shows examples of the argillite, basalt and one type of sandstone (S3) rock core sample used in this research. Due to some issues, the rock quality designation (RQD) test could only be performed on these three types of samples. The basic procedure of the RQD test is according to the ASTM D6032. Therefore, only the calculation procedure is provided in this section.

For argillite, three rock cores were used, the total length of which was 2974 mm:

- For the first rock core (length = 1010 mm), the total length of the pieces which were larger than 100 mm was 1008 mm.
- For the second rock core (length = 924 mm), the total length of the pieces larger than 100 mm was 751 mm.
- For the third rock core (length = 1040 mm), the total length of the pieces larger than 100 mm was 608 mm.

After these lengths were calculated, the depth of the rock core was noted down and the RQD can be calculated as the ratio of the number of joints over the length of the core.
The rock porosity and density tests were conducted according to AS 4133.2.1.2-2005 to determine the porosity and the dry density of the rock samples. The procedure of the test is as follows:

- Collect at least ten different samples for one type of rock, wash all the samples and soak them in water for an hour.
- Saturate all the samples in a bowl and immerse them under water. Vacuum the sample at not less than 800 Pa for at least 1 hour to make sure all the trapped air has been removed.
- Determine the mass of the weight of the basket which is used to contain the rocks for weighing the samples underwater.
• Transfer the rock samples underwater into the basket which is already immersed under water. From this, the saturated submerged mass can be determined.

• Remove all samples from the water bath, gently clean the surface water of these rock samples and weigh all samples again. Determine the saturated and surface-dry sample weights.

• Place all the wet samples in an oven at 105°C and at least 24 h when they all dry sufficiently, weight them again to get the dry weight.

• Note down the room temperature and the bath temperature.

Figure 3-2 shows the equipment used for the porosity test. The density and porosity were obtained by using the formula provided by AS 4133.2.1.2-2005. All results are provided in the following chapters.

Figure 3-2 (a) Drying oven (b) Vacuum saturation equipment (c) An immersion bath and a wire basket

For the porosity, the calculation process is as follows:

(a) Calculate the saturated-surface-dry mass \(m_{\text{sat}}\) of the sample:

\[ m_{\text{sub}} = m_{\text{ss}} - m_{\text{bs}} \]
Where:

$m_{\text{sub}}$: saturated submerged mass, in grams.

$m_{\text{ss}}$: saturated submerged mass of the basket plus sample, in grams.

$m_{\text{bs}}$: saturated submerged mass of the basket alone, in grams.

(b) Calculate the saturated-surface-dry mass ($m_{\text{sat}}$) of the sample:

$$m_{\text{sat}} = m_{\text{sd}} - m_{\text{c}}$$  \hspace{1cm} 3.2

Where:

$m_{\text{sat}}$: saturated surface-dry mass, in grams.

$m_{\text{sd}}$: saturated surface-dry mass of the sample plus container, in grams.

$m_{\text{c}}$: mass of the container, in grams.

(c) Calculate the grain mass ($m_{\text{g}}$) of the sample:

$$m_{\text{g}} = m_{\text{cd}} - m_{\text{c}}$$  \hspace{1cm} 3.3

Where:

$m_{\text{g}}$: grain mass, in grams.

$m_{\text{cd}}$: mass of the dried sample and container, in grams.

$m_{\text{c}}$: mass of the container, in grams.

(d) Calculate the bulk volume ($V_{b}$) of the sample:

$$V_{b} = \frac{(m_{\text{sat}} - m_{\text{sub}})}{\rho_{w}}$$  \hspace{1cm} 3.4

Where:

$V_{b}$: bulk volume, in cubic centimetres.

$m_{\text{sat}}$: saturated surface-dry mass, in grams.

$m_{\text{sub}}$: saturated submerged mass, in grams.
\( \rho_w \): density of water at temperature \( t \) °C, in grams per cubic centimetre.

(e) Calculate the pore volume \( (V_p) \) of the sample:

\[
V_p = \frac{(m_{\text{sat}} - m_g)}{\rho_w}
\]

Where:

\( V_p \): pore volume, in cubic centimetres.

\( m_{\text{sat}} \): saturated surface-dry mass, in grams.

\( m_g \): grain mass, in grams.

\( \rho_w \): density of water at temperature \( t \) (°C), in grams per cubic centimetre.

(f) Calculate the porosity \( (n) \) of the rock sample:

\[
n = \frac{V_p}{V_b} \times 100
\]

Where:

\( n \): porosity of rock, in percent.

\( V_p \): pore volume, in cubic centimetres.

\( V_b \): bulk volume, in cubic centimetres.

(g) The dry density \( (\rho_d) \) of the rock sample:

\[
\rho_d = \frac{m_g}{V_b}
\]

Where:

\( m_g \): grain mass, in grams.

\( V_b \): bulk volume, in cubic centimetres.
3.5 XRD test

An XRD test is an analytical technique which is a clipping method. This technique is mainly used for the identification of a crystalline material and provides information of the unit cell. For this research, the XRD result can provide a fresh view of the propagation of the cracks. Figure 3-3 shows the normal set of XRD equipment.

![XRD test machine](https://serc.carleton.edu)

**Figure 3- 3 XRD test machine (https://serc.carleton.edu)**

3.6 Optical Microscope

A Nikon SMZ745T optical microscope (Figure 3-4) was used and connected to the computer to capture images and simplify the process of digital imaging. This optical microscope uses the Greenough optical system, which can zoom to 7.5x and has a total magnification range from 3.35x to 300x. The procedure of using this microscope is as follows:

- Fix the rock sample on a piece of black cardboard with a blue tag in between.
- Adjust the microscope manually by using the focus knob and check with the eyepiece.
- Open the camera switch and adjust the view by using the knob and check the image the computer screen.
Switch the camera to high quality, freeze the image and take a screenshot via the microscope’s software.

3.7 Barton’s comb and tilt test.

A Barton’s comb test (Fig. 3-5) was used to measure the joint roughness coefficient (JRC) (Barton and Choubey, 1977; ISRM, 1978; Barton and Bandis, 1980). A JRC value obtained by a Barton’s comb test can be used both in the laboratory and the field. The purpose of this device is to describe the sample’s surface roughness condition so that it may be taken into account during the preliminary design phase of a project involving rock engineering. If more research is required, the shear strength of the joint can be estimated using other criteria.

As seen in Figure 3-5, this comb is made by several steel wires that can properly touch the joint surface of a rock. The contour of the rock joint surface may then be drawn on
the paper, and the JRC for the range of samples can be obtained by comparing it to the roughness profile created by Barton and Choubey (1977). The test procedure is as follows:

- Align the steel wires by pressing the device on a plane surface.
- Hold the Barton’s comb parallel to the wall of the core and press the Barton’s comb against the rock joint; repeat this process until the most rugged profile is obtained.
- Measure the asperity height of the profile.
- Depict the profile of each rock samples on the paper and compare to the standard profile in Appendix 1 and 2.
- Report the JRC value of this sample.
- Repeat the steps above for each rock sample.

![Barton’s comb and the measurement of the roughness profile on a rock sample](image)

Figure 3-5 Barton’s comb and the measurement of the roughness profile on a rock sample

Several specimens were prepared by sanding them. The sliding surfaces of the specimens were manually smoothed with sandpaper. These samples were used to determine the basic friction angle of clean, smooth sandstone and argillite discontinuities.
3.8 Point load test

In order to test an irregularly shaped sample’s strength, a point load test was applied in this study. Compared with the regular unconfined compression test, the point load test does not require the sample’s geometry to be as rigorous as the UCS test does. By following AS 4133.4.1-2007 “Methods of testing rocks for engineering purposes, Method 4.1: Rock strength test-determination of point load strength index,” for different geometries of samples, the different equivalent core diameters can be determined (Figure 3-7).
Chapter 3

Figure 3-7 Shape proportions and equivalent core diameter of test specimens (AS 4133.4.1)

For cracking in different directions of the testing sample, there are two methods of calculation. For the diametral test, the calculation is as follows:

\[ I_s = \frac{P \times 1000}{D^2} \]  

Where:

- \( I_s \): uncorrected point load strength, in MPa.
- \( P \): load at failure, in kN.
- \( D \): platen separation, in mm.

For the axial, block and irregular lump samples, the relevant calculations are as follows:

\[ I_s = \frac{P \times 1000}{D^2} \]  

Where:

- \( I_s \): uncorrected point load strength, in MPa.
- \( P \): load at failure, in kN.
- \( D \): platen separation, in mm.
\[ D_e = \sqrt{\frac{4 \times A}{\pi}} \]  

3.10

Where:

I<sub>s</sub>: uncorrected point load strength, in MPa.

P: load at failure, in kN.

D<sub>e</sub>: equivalent core diameter, in mm.

A: minimum cross-sectional area of the plane through the platen points, in square millimetres.

By using the uncorrected point load strength, the point load strength index can be obtained by:

\[ I_s(50) = I_s \times \left( \frac{D_e}{50} \right)^{0.45} \]  

3.11

Where:

I<sub>s</sub>: uncorrected point load strength, in MPa.

D: platen separation or equivalent core diameter, in mm.

I<sub>s(50)</sub>: point load strength index.

Thus, by using this point load strength index with the relevant ratio, for those rock samples that did not meet the UCS testing requirements, the unconfined compression strength can be calculated.

3.9 Unconfined compression test

After the basic experimentation to determine the sample’s mechanical properties, all samples were cut to have a height of 100 mm and a diameter of 50 mm. Pre-existing
discontinuities (cracks) were made on the top and bottom of the specimens by means of an electric grinder. For all types of sandstone (S1, S2, and S3), cracks either 1 cm or 2 cm deep were made (Figure 3-8). However, for the argillite and basalt, it was rather difficult to create pre-existing cracks deeper than 1 cm without fracturing the whole specimen. For this reason, both the argillite and basalt samples contained pre-existing cracks of only 1 cm deep. The width of cracks for each type of rock was 3 mm.

The infill material for all types of rock was moist sand with a friction angle of 45° and water content of 15%. In addition, moist clay was used as the infill material for all types of sandstone as there was a sufficient number of sandstone specimens available to complete this series of tests. The clay had a water content of 25% and an unconfined compressive strength of 98 kPa. For each type of rock, the infill was placed in the pre-existing crack in two layers and each layer was gently tamped to achieve an overall density of 1.9 g/cm³ (sand infill) and 2.0 g/cm³ (clay infill).

To observe the propagation of the crack, all the specimens underwent a uniaxial compressive strength (UCS) test. The UCS test machine is shown in Figure 3-9. All the tests were recorded by a high-speed camera to capture the failure behaviour, which gave a much
clearer and more accurate image of the propagation than an ordinary speed camera. The procedure for the UCS test was as follows:

- Place the sample in the centre of the compression plate for best results
- Set the program only to the compression test because the machine can perform other tests such as tensile strength and modulus of elasticity
- According to Australian standards, the input applied compression load rate is set at 0.5 kN / sec.
- Use a rubber cap to cover the top of the sample.

The rubber capping allows the compressive load to be evenly distributed over the circumferential area of the sample. However, due to the small size of the testing sample, instead of using rubber cap, all the samples were capped by gypsum at both sides.

3.10 Direct shear test

As it can determine the ultimate shear strength under unconfined circumstances, the direct shear test is crucial for determining the shear strength of a jointed rock. The discontinuities in the rock mass, which are the weakest section of the rock mass, are where
rock collapse occurs most frequently in shear rocks. Indraratna and others (2008) noted that the infill joint may, however, end up being the weakest section of a rock mass if the joint surface loses touch as a portion of the surface rock weathers and becomes infill. By applying the method devised by Barton and Choubey (1977), the maximum shear strength of a rock discontinuity to cause an onset movement may be determined. The relevant calculation is as follows:

\[
\tau = \sigma_n \tan \left( \phi_r + JRC \log_{10} \left( \frac{JCS}{\sigma_n} \right) \right)
\]

Where:

\( \tau \): shear stress, in megapascals.

\( \sigma_n \): normal stress, in megapascals.

\( \phi_r \): residual friction angle, in degrees.

\( JRC \): joint roughness coefficient.

\( JCS \): joint wall compressive strength which is same as UCS.

In the laboratory, the direct shear test can be performed on both clean jointed rock specimens and the specimens with clay infill, using a portable shear box.

All shear tests in this investigation were conducted using a portable shear box (Fig. 3-10) SL 900 on loan from a geotechnical engineering laboratory, and they were done in accordance with ASTM D5607-02 (Muralha et al. 2014).
The Barton’s comb test was used to capture each specimen’s roughness profiles before the direct shear test samples (Figure 3-11) were made. Each specimen’s top and bottom were marked, and a cement encasing mixture was made for the initial pour into the moulds used for the direct shear test, where the sample would be half immersed. After this a period of at least 24 hours passed for the mixture to be cured. The other half of the sample was turned upside down and placed inside the new mould once the material had solidified sufficiently to maintain its shape after a second pour of the mixture into a different mould. The sample was taken out of the moulds and placed in a suitable area to cure after the material had set. After this preparation, the shear testing procedure was conducted as follows:

- Remove the upper housing of the apparatus.
- Place the sample inside the housing and carefully place the upper housing over the sample.
- Apply a shear load at a uniform rate to the sample using the hand pressure pump.
Begin shearing by applying the shear stress at a uniform rate to the sample using the other hand pressure pump. The dial gauge must be monitored to ensure that a reasonably uniform rate of displacement is achieved.

- Record the level of normal stress and the ultimate shear stress.
- Continue shearing beyond this peak so that a reliable residual shear strength can be obtained.
- Repeat the process for the rest of samples.

![Image of direct shear sample in casting capsule]

**Figure 3- 11 Direct shear sample in casting capsule**

3.11 Boundary-effect and size-effect

Based on the study of Krayani et al. (2009) and Yu et al. (2010), in most experimental and theoretical studies, boundary-effects and scale-effects always exist. However, to the peak value of both UCS strength and shear strength, the difference due to the size effect is not relative (Tang et al., 1996)

As for the boundary effect of the new cut in the numerical model, the UDEC can generate a new boundary at the contact surface, which also shows the difference between the DEM and FEM methods. This means if the new cuts have been assigned their property appropriately, the boundary effects can be minimised.
CHAPTER 4: MATERIALS GEOLOGY

From a geotechnical business company on the Gold Coast, several kinds of rock samples were obtained. The locations from which these rock samples were taken are shown in Figure 4-1. These regions show different rock units. There were four different kinds of sandstone. The argillite and Sandstone 3 are from the Neranleigh-Fernvale Beds, the basalt is from the lavas of the Tweed Volcano, and Sandstone 1 is from the Beaudesert Beds. Sandstone 2 and 4 is from the Marburg Formation.

Figure 4-1 Geological map of the GOLD Coast area (Kim et al. 2013)

The explanations that follow provide more information on the Gold Coast’s geological formation. The Neranleigh-Fernvale Beds, the Bundamba group sedimentary strata, and the Lamington Volcanics are the three primary formations of geological units in
the Tertiary, Triassic, and Carboniferous periods in the Gold Coast region, respectively, according to Whitlow (2000).

The vast Neranleigh-Fernvale Beds, which appeared some 290 million years ago and form around 85% of the hard rock area, are what make up the Gold Coast (Whitlow, 2000). This region has large areas of argillite, chert, greenstone and oolitic labile arenite (Shokouhi et al., 2013 and Kim et al., 2015). Deep sea sediments and the continental shelf are the sources of the Neranleigh-Fernvale Beds (Willmott, 1983). In this study, fresh argillite has been selected to represent this area. Fresh argillite was of relatively high unconfined compressive strength (UCS) which is about 39.4 MPa (Vidana Pathiranagei et al., 2023), and also the porosity values of the argillite is less than 3%.

The Woogaroo and Marburg subgroups constitute the Bundamba group, which was produced towards the end of the Triassic epoch (Willmott, 1981 and Wells and O’Brien, 1994). Sandstone dominates the Marburg Formation, which was formed between 220 and 160 million years ago. These rocks can be found in a large area that stretches from Canungra to Jimboomba in the north and to Rathdowney in the south (Willmott, 1983). Sandstones from these areas generally contained various clay matrix and rock fragments, which estimated that there is up to 60% of quartz of rock composition with small amount of clay minerals. The average porosity of sandstone from this area is about 9% and UCS about 13.9 MPa (Vidana Pathiranagei et al., 2023).

The Beaudesert Beds, which are composed of soft sandstone and siltstone that are interbedded, black carbon-bearing shale, and poorly consolidated conglomerate, originated between 55 and 45 million years ago (Willmott and Monteith, 1999). Tectonic activity led to the formation of the Tweed Volcano about 23 million years ago (Willmott and Monteith, 1999).
CHAPTER 4

The volcano flows in Southeast Queensland created the alkaline and subalkaline lava. As a result, the Lamington Volcanics’ igneous rocks were created (Duggan and Mason 1978), and today’s Springbrook and the Numinbah Valley regions of the Gold Coast still contain volcanic rocks, including basalt and rhyolite (Wilkinson, 1968). The fresh basalt shows a dark black colour, high strength (UCS is about 83.7 MPa) and low porosity value (2.6%). Even though this study is mainly focused on fresh samples, it is necessary to find out the degree of weathering based on ISRM (1981).
Chapter 5

EFFECTS OF PRE-EXISTING CRACKS AND INFILLING ON STRENGTH OF NATUREAL ROCKS-CASES OF SANDSTONE, ARGILLITE AND BASALT

Statement of contribution to co-authored published paper

This chapter includes a co-authored paper. The bibliographic details of the co-authored paper, including all authors, are:


Note that the format of this paper has been changed according to the guidelines of the thesis. My contribution to the paper involved: literature review, experimental design and set up, experimental data collect and analyses, summarize and discuss the test results, paper writing and responding to reviewers.

(Signed) _________________________________ (Date) 15/12/2022
Chen Cui

(Countersigned) ___________________________ (Date) 15/12/2022
Principal Supervisor: A/P Gratchev Ivan
CHAPTER 5: EFFECT OF PRE-EXISTING CRACKS AND SOIL INFILL ON STRENGTH OF SANDSTONE, ARGILLITE AND BASALT

Abstract:

This study aims to examine the influence of pre-existing discontinuities on strength of four natural rocks with different origin. A series of unconfined compression tests were performed on specimens of sandstone, argillite and basalt that contain open and filled cracks. It was found that the presence of cracks tends to decrease the overall strength for all studied rocks; however, the magnitude of strength reduction is related to the property of rock. The higher strength decrease was observed for the relatively harder argillite and basalt, compared to the softer sandstone. It was also found that the infill material could increase the strength of rock specimens while the amount of strength gain depended on the characteristics of filled material.

Key words: crack; strength; infill; laboratory test
5.1 Introduction

In geotechnical engineering, there are several types of discontinuities associated with shear zones, bedding planes and faults that can affect the strength and stiffness of rock mass (Shen, 1995; Eberhard et al., 1998). In the last decades, a good deal of research has been performed to study the effect of cracks on the strength of rock and rock-like material with a focus on the process of crack initiation, propagation and coalescence of pre-existing cracks (Eberhard et al., 1998; Yang et al., 2012; Xu et al., 2013; Nikolic et al., 2014; Shang et al., 2016; Cheng et al., 2016; Roy et al., 2016). As a result, several uniaxial tests (Ramamurthy and Arora, 1994; Shen et al., 1995; Sagong and Bobet, 2002; Xu et al., 2013; Lisjak et al., 2014; Wong and Zhang, 2014; Yin et al., 2014; Zhou et al., 2014; Nikolic and Ibrahimbegovic, 2015; Cao et al., 2015; Gratchev et al., 2016; Zhou et al., 2018), biaxial tests (Bobet and Einstein, 1998), and shear or tensile tests (Gehle and Kutter, 2003; Indraratna et al., 2010; Indraratna et al., 2014; Shang et al., 2018)) have been conducted on rock-like material with pre-existing multiple either closed or opened cracks. These studies show that the crack propagation mostly initiates from the pre-existing cracks and grows into rock bridge, eventually producing new discontinuities that would connect the pre-existing ones (Gehle and Kutter, 2003; Yin et al., 2014; Cao et al., 2015).

As the process of crack initiation and propagation appears to be rather complex by itself, the majority of previous studies have employed rock-like material, making it easier to produce specimens with certain crack patterns. Although such studies have significantly advanced our knowledge in this area, the rock-like material is still rather different from natural rocks as it does not represent the inhomogeneity (such as voids, macro cracks, etc.) which is common for natural rocks (Zhang, 2010). In addition, only limited research was conducted to investigate the effect of infill on rock strength even though rock discontinuities
are often filled with soil material in the field. Rock joints may be filled with either crystalline material (e.g., quartz, calcite) or soft weathering products (i.e., gouge in the form of sand and/or clay). While the former can lead to crack sealing, a mechanism that can contribute to the overall rock strength (Ramsay, 1980), the latter can decrease the rock mass strength due to the infill low friction properties (Indraratna et al., 2005). The effect of infill is especially of great importance when rock mass can potentially fail on filled joints in shear, resulting in landslide movements. Several studies have been conducted in the past few decades to better understand the effect of soil-like infill such as sand (Tulinov and Molokov, 1971) and clay (Kanji, 1974; Ladanyi and Archambault, 1977; Barla et al., 1985) on the shear strength of jointed rocks. However, no systematic studies have been conducted up-to-date to investigate the effect of infill on the unconfined compressive strength (UCS) of natural rock although UCS is one of the most important rock strength parameters that is used in all rock mass classification systems (Gratchev et al., 2019). This work seeks to shed light on this issue and provide new data on the unconfined strength of natural rocks with different types of infill. In particular, this study investigates the effect of pre-existing cracks on strength of four natural rocks and clarify the effect of infill material such as sand and clay on crack development and failure patterns. This paper presents and discusses the obtained results.

5.2 Experimental program

5.2.1 Rock used

Core samples of fresh sandstone, argillite and basalt from the Southeast Queensland, Australia were provided for this study by local geotechnical companies. Two types of sandstone, namely S1 and S2 were used. S1 was grey, relatively weak rock with high porosity (n=29.1%) and a relatively low unconfined compressive strength (UCS) of 11.5 MPa. S2 had an average porosity value of 8.4%, and the unconfined compressive strength
of 47.5MPa. The argillite and basalt specimens were of low porosity: about 4.9% and 1.4%, respectively. The properties of each type of rock as well as its mineral composition are given in Table 5-1.

**Table 5-1 Rock properties**

<table>
<thead>
<tr>
<th>Name</th>
<th>Porosity (%)</th>
<th>Density (g/cm³)</th>
<th>UCS (MPa)</th>
<th>Major minerals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone S1</td>
<td>29.1</td>
<td>1.78</td>
<td>11.5</td>
<td>Quartz, Calcite, Kaolinite</td>
</tr>
<tr>
<td>Sandstone S2</td>
<td>8.4</td>
<td>2.57</td>
<td>47.5</td>
<td>Quartz, Feldspar, Muscovite</td>
</tr>
<tr>
<td>Argillite</td>
<td>4.9</td>
<td>2.55</td>
<td>39.4</td>
<td>Quartz, Feldspar, Kaolinite, Calcite, Illite, Muscovite</td>
</tr>
<tr>
<td>Basalt</td>
<td>1.4</td>
<td>2.72</td>
<td>83.7</td>
<td>Pyroxene, Olivine, Analcime, Quartz, Illite, Plagioclase</td>
</tr>
</tbody>
</table>

5.2.2 Testing program

All rock specimens tested in unconfined compression had a height of 100 mm and diameter of 50 mm (Figure 5-1). Pre-existing discontinuities (cracks) were made on top and bottom of the specimen by means of electric grinder. For two types of sandstone (S1, S2), cracks of either 1 cm or 2 cm deep were made. However, for the argillite and basalt, it was rather difficult to create pre-existing cracks deeper than 1 cm without fracturing the whole specimen. For this reason, both argillite and basalt contained pre-existing cracks of only 1 cm deep. The width of cracks for each type of rock was 3 mm (Figure 5-1).

The infill material for all four rock types was moist sand (classified as poorly-graded sand, according to USCS) with a friction angle of 35° and water content of 15%. In addition, moist clay (classified as CL, according to USCS) was used as infill material for two types of...
sandstone as there were sufficient number of sandstone specimens available to complete this series of tests. The clay had water content of 25% and unconfined compressive strength of 98 kPa. For each type of rock, the infill was placed in the pre-existing crack in two layers and each layer was gently tamped to achieve the overall density of 1.9 g/cm³ (sand infill) and 2.2 g/cm³ (clay infill).

![Diagram of sample size: (a) 10 mm crack depth, (b) 20 mm crack depth](image)

Figure 5-1 Sample size: (a) 10 mm crack depth, (b) 20 mm crack depth

For each specimen, a compression load was applied with an increment of 0.5 kN/s until the specimen failed (AS 1141-1996). A high-speed camera was used to record the behaviour of the specimen throughout the test. For each experimental setup, the test was repeated at least 3 times and the average value was used for analysis. The summary of test results is given in Table 5-2.
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Rock types</th>
<th>Crack properties</th>
<th>Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sandstone S1</td>
<td>Intact (no crack)</td>
<td>13.6</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td>10.9</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td>10.6</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td>11</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td>10.4</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>1 cm crack</td>
<td>8.7</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td>9.1</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td>8.1</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>2 cm crack</td>
<td>8.7</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td>9.7</td>
</tr>
<tr>
<td>11</td>
<td>Sandstone S1</td>
<td>1 cm crack with sand infill</td>
<td>14.9</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>1 cm crack with sand infill</td>
<td>13.8</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td></td>
<td>14.5</td>
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<td>13.1</td>
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<tr>
<td>15</td>
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<td>2 cm crack with sand infill</td>
<td>15.9</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td>13.9</td>
</tr>
<tr>
<td>17</td>
<td></td>
<td>1 cm crack with clay infill</td>
<td>16.1</td>
</tr>
<tr>
<td>18</td>
<td></td>
<td>1 cm crack with clay infill</td>
<td>15.5</td>
</tr>
<tr>
<td>19</td>
<td></td>
<td></td>
<td>15.8</td>
</tr>
<tr>
<td>20</td>
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<td></td>
<td>14.7</td>
</tr>
<tr>
<td>21</td>
<td></td>
<td>2 cm crack with clay infill</td>
<td>17.5</td>
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<tr>
<td>22</td>
<td></td>
<td></td>
<td>12.4</td>
</tr>
<tr>
<td>No.</td>
<td>Material</td>
<td>Condition</td>
<td>Value</td>
</tr>
<tr>
<td>-----</td>
<td>----------</td>
<td>-----------</td>
<td>-------</td>
</tr>
<tr>
<td>23</td>
<td>Sandstones S2</td>
<td>Intact (no crack)</td>
<td>48.9</td>
</tr>
<tr>
<td>24</td>
<td>Sandstones S2</td>
<td></td>
<td>44.6</td>
</tr>
<tr>
<td>25</td>
<td>Sandstones S2</td>
<td></td>
<td>48.9</td>
</tr>
<tr>
<td>26</td>
<td>Sandstones S2</td>
<td>1 cm crack</td>
<td>40.8</td>
</tr>
<tr>
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<td>Sandstones S2</td>
<td>1 cm crack</td>
<td>41.8</td>
</tr>
<tr>
<td>28</td>
<td>Sandstones S2</td>
<td></td>
<td>40.8</td>
</tr>
<tr>
<td>29</td>
<td>Sandstones S2</td>
<td></td>
<td>32.6</td>
</tr>
<tr>
<td>30</td>
<td>Sandstones S2</td>
<td>2 cm crack</td>
<td>32.1</td>
</tr>
<tr>
<td>31</td>
<td>Sandstones S2</td>
<td></td>
<td>32.7</td>
</tr>
<tr>
<td>32</td>
<td>Sandstones S2</td>
<td></td>
<td>39.7</td>
</tr>
<tr>
<td>33</td>
<td>Sandstones S2</td>
<td>1 cm crack with sand infill</td>
<td>36.2</td>
</tr>
<tr>
<td>34</td>
<td>Sandstones S2</td>
<td></td>
<td>42.8</td>
</tr>
<tr>
<td>35</td>
<td>Sandstones S2</td>
<td></td>
<td>31.0</td>
</tr>
<tr>
<td>36</td>
<td>Sandstones S2</td>
<td>2 cm crack with sand infill</td>
<td>33.1</td>
</tr>
<tr>
<td>37</td>
<td>Sandstones S2</td>
<td></td>
<td>31.9</td>
</tr>
<tr>
<td>38</td>
<td>Sandstones S2</td>
<td></td>
<td>45.9</td>
</tr>
<tr>
<td>39</td>
<td>Sandstones S2</td>
<td>1 cm crack with clay-infill</td>
<td>43.3</td>
</tr>
<tr>
<td>40</td>
<td>Sandstones S2</td>
<td></td>
<td>45.9</td>
</tr>
<tr>
<td>41</td>
<td>Sandstones S2</td>
<td></td>
<td>45.9</td>
</tr>
<tr>
<td>42</td>
<td>Sandstones S2</td>
<td>2 cm crack with clay infill</td>
<td>42.5</td>
</tr>
<tr>
<td>43</td>
<td>Sandstones S2</td>
<td></td>
<td>39.7</td>
</tr>
<tr>
<td>44</td>
<td>Argillite</td>
<td>Intact (no crack)</td>
<td>42.3</td>
</tr>
<tr>
<td>45</td>
<td>Argillite</td>
<td></td>
<td>36.5</td>
</tr>
</tbody>
</table>
### 5.3 Results and discussion

#### 5.3.1 Tests result obtained for argillite and basalt

Typical results from unconfined compression tests are given in Figure 2 for argillite and Figure 5-2 for the basalt specimens.

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>46</td>
<td></td>
<td>39.3</td>
</tr>
<tr>
<td>47</td>
<td></td>
<td>28.8</td>
</tr>
<tr>
<td>48</td>
<td>1 cm crack</td>
<td>30.9</td>
</tr>
<tr>
<td>49</td>
<td></td>
<td>30.5</td>
</tr>
<tr>
<td>50</td>
<td>1 cm crack with sand infill</td>
<td>31.1</td>
</tr>
<tr>
<td>51</td>
<td>1 cm crack with sand infill</td>
<td>31.5</td>
</tr>
<tr>
<td>52</td>
<td></td>
<td>33.2</td>
</tr>
<tr>
<td>53</td>
<td>Intact (no crack)</td>
<td>86.4</td>
</tr>
<tr>
<td>54</td>
<td></td>
<td>86.9</td>
</tr>
<tr>
<td>55</td>
<td></td>
<td>77.8</td>
</tr>
<tr>
<td>56</td>
<td>Basalt</td>
<td>68.3</td>
</tr>
<tr>
<td>57</td>
<td>1 cm crack</td>
<td>70</td>
</tr>
<tr>
<td>58</td>
<td></td>
<td>69.6</td>
</tr>
<tr>
<td>59</td>
<td></td>
<td>100.2</td>
</tr>
<tr>
<td>60</td>
<td>1 cm crack with sand infill</td>
<td>113.1</td>
</tr>
<tr>
<td>61</td>
<td></td>
<td>99.7</td>
</tr>
</tbody>
</table>
For the argillite, the maximum unconfined compressive strength of about 40 MPa was observed for the intact specimen while the lowest strength of 30 MPa was recorded for the specimen with open cracks. The basalt was the hardest rock with UCS of 76 MPa for the intact specimen (Figure 5-3). However, the greatest strength was observed for the specimen with 1 cm filled cracks.
As can be seen in Figs 5-2 and 5-3, the 1 cm open cracks tend to decrease the overall strength of argillite and basalt by 23% and 17%, respectively, while the sand infill results in strength increase. For the argillite (Figure 5-2), the strength increase due to the infill was observed to be rather small (about 6%). However, the basalt specimens (Figure 5-3) with the filled pre-existing cracks exhibited a significant strength increase from an average of 69.3 MPa (open crack) to as high as 104.3 MPa (filled crack with sand), even exceeding the strength of intact rock (an average of 83.7 MPa). Such changes in strength can be attributed to the stress-strain behavior of the basalt specimens (Figure 5-4). The specimens with open cracks exhibited a distinct brittle behavior with sudden failure under high loads while the specimens with sand infill were able to undertake larger deformations (greater strain), leading to greater stresses at failure. The summary of the stress-strain data at failure obtained for the basalt specimens with open and filled cracks is given in Figure 5-4. It can be inferred from this figure that the failure of intact specimens and the specimens with open cracks occurred at smaller strain values while the specimens with filled cracks failed at relatively greater strains and stresses. On the contrary, for the softer argillite, the greater stresses and strains were measured for the intact specimens (Figure 5-5).
5.3.2 Tests result obtained for sandstone

The test results obtained for two types of sandstone with the pre-existing open cracks of 1 cm and 2 cm deep are given in Figure 5-6 and 5-7. The lab data obtained for S1 (Figure 5-6), indicated that the specimens with 2 cm cracks (no infill) produced a lower strength (UCS=8.8 MPa) than both the intact specimens (UCS=11.5 MPa) and the specimens with 1 cm crack (UCS=9.4 MPa). The same tendency was observed for S2, in which the greatest strength was obtained for the intact specimens (on average of 47.5 MPa) while the lowest strength (34.5 MPa) was measured for the specimens with the longer open crack of 2 cm.

Compared to the specimens with open cracks, the infill material (either sand or clay) generally increased the rock strength, while in some cases, it even exceeded the strength of intact rock. The effect of infill on UCS was more pronounced for S1, which can be attributed to the fact that the infill formed ‘inclusions’ which were slightly denser than the rock density (1.78 g/cm³). The inclusion of sand (density of 1.9 g/cm³) or clay (density of 2.2 g/cm³)
seemed to add some additional strength of about 35% and 41%, respectively. Compared to the sand infill, the denser clay infill appears to be the main reason for the slightly higher values of UCS obtained for both S1 and S2 specimens whose cracks were filled with the clay. It is also noted that the effect of infill on UCS becomes less significant for the denser S2 (average density of 2.57 g/cm$^3$), which can be clearly seen in Fig.5.7.

It is noted that for S1, regardless of the crack length and infill type, greater strength correlates with greater strain at failure. For S2, this correlation is not very clear as the specimens with higher UCS does not necessarily fail at greater strain.

![Figure 5-6 Summary of stress-strain distribution of S1: (a) S1 with 1 cm crack, (b) S1 with 2 cm crack](image)
5.3.3 Typical failure patterns

Visual observations during testing and analysis of high-speed camera footage revealed similarities in the failure pattern of different rock types with pre-existing cracks. Figure 5-8 to 5-12 summarize the typical failure patterns with the initiation and propagation (from left to right) of cracks observed for specimens with different crack characteristics.
For most intact specimens, regardless of the rock type, newly-developed cracks propagated through the whole specimen resulting in failure as schematically shown in Figure 5-8. Unlike other rocks, the hard basalt exhibited a distinct brittle behavior under high stresses.

The specimens with pre-existing cracks failed in different fashions. It was found that for the both types of sandstone and argillite, new cracks typically formed near the pre-existing discontinuities and then propagated through the specimen, leading to failure (Figure 5-9). For the basalt specimen with 1 cm open crack, it was rather difficult to analyze the crack propagation pattern as most of specimens failed in a sudden and brittle manner. It is also noted that there were a few cases where newly-developed cracks in S1 and S2 did not initiate from the existing discontinues. Compared to the available literature, the failure patterns of natural rocks appear to be similar to what was reported for rock-like specimens (Gratchev et al., 2016).

As for the specimens with filled cracks, the typical failure patterns propagated as shown in Figure 5-10 and 5-11. For 1 cm cracks, newly-developed discontinues originated from one of the existing cracks (not from both) and then propagated through the sample causing failure.

A special case was observed for S1, 1 cm filled cracks with clay (Figure 5-12), in which new crack developed at the edges of specimen and coalesced, referring in failure.
Figure 5-8 Typical failure patterns of intact sample

Figure 5-9 Typical failure patterns of specimen with open cracks

Figure 5-10 Typical failure patterns of specimen with 1 cm filled cracks
5.3.4 Effect of infill on rock behaviour

Results from UCS tests and observed failure patterns indicate that the effect of infill on rock strength is related to rock origin and its properties. Compared to previous studies on the shear behavior of filled rock joints (De Toledo et al., 1993; Indraratna et al., 2005) in which the effect of infill was mainly seen in decreasing the overall shear strength, this study shows that the infill can greatly influence the failure mode and add to overall rock strength (UCS). It is noted that unlike the aforementioned studies that involve the shear properties of infill, the current study focuses on the unconfined compressive strength of jointed rock.
where the infill can only contribute to rock strength by filling the empty space of the pre-existing crack.

It was observed that for most specimens regardless of the rock type, newly-developed discontinuities originated from the tip of the pre-existing cracks (Figure 5-9) while for the specimens with filled cracks, the new discontinuities initiated from the side of the specimen (Figure 5-10). In addition, for both jointed sandstones (S1 and S2) filled with clay, the failure pattern (Figure 5-12) was very similar to the one which was observed for the intact specimens (Figure 5-8). Thus, changes in the failure patterns may be associated with the observed difference in rock strength.

This research also shows that the effect of infill depends on the rock origin. For example, for the very soft S1, the infill served as additional reinforcement as the density of sand (1.9 g/cm$^3$) and clay (2.2 g/cm$^3$) infill was significantly greater than the rock density (1.78 g/cm$^3$). This resulted in greater UCS values which were obtained for the jointed rock specimens with the infill. However, for the argillite and S2 specimens with greater rock density of 2.55 g/cm$^3$ and 2.57 g/cm$^3$, respectively the influence of infill was less pronounced, compared to S1. For the basalt specimens with a defined brittle behavior, the influence of infill was found to be of different nature. As shown in Fig.5-4, the infill provided additional deformation under loads which resulted in greater strength.

5.4 Conclusions

In this work, the effect of pre-existing open and filled cracks on two types of sandstone, argillite and basalt was studied through a series of unconfined compression tests. Based on the obtained results, the following major conclusions can be drawn:
For all rock types, the pre-existing open cracks decreased the unconfined compressive strength. The higher strength reduction was observed for the relatively harder rocks such as argillite and basalt.

The filled pre-existing cracks were found to increase the unconfined compressive strength (UCS) of all studied rocks. It was found that the denser infill produced greater increases in UCS.

The failure process was driven by newly-developed discontinuities that initiated from the pre-existing cracks. This finding seems to agree with previous data obtained for rock-like specimens.
Chapter 6

CHANGE IN JOINT SURFACE ROUGHNESS OF TWO NATURAL ROCKS DURING SHEARING

Statement of contribution to co-authored published paper

This chapter includes a co-authored paper. The bibliographic details of the co-authored paper, including all authors, are:


Note that the format of this paper has been changed according to the guidelines of the thesis, and supplementary experimental details which were not included in this paper for publication now have been provided in “APPENDIX 1” for references. My contribution to the paper involved: literature review, experimental data analysis, discussion of test results and development of the analytical study, paper writing and responding to reviewers.

(Signed) _______________________________ (Date) ___________
Chen Cui

(Countersigned) _____________________________ (Date) ___________
Principal Supervisor: A/Professor Gratchev Ivan

(Countersigned) _____________________________ (Date) ___________
Mr Matthew Chung
(Countersigned) ____________________________ (Date) 14/12/2022

Dr Dong Hyun Kim
CHAPTER 6: CHANGES IN JOINT SURFACE ROUGHNESS OF TWO NATURAL ROCKS DURING SHEARING

ABSTRACT:

Tamborine Mountain is one of the rock slopes located at Gold Coast area in Australia which always happened a failure after a heavy rainfall. When the slop failure happened, the shear strength occurred between the discontinued joints. Joint roughness coefficient (JRC) has been performed to modify the discontinued joint roughness and the accuracy of JRC value is also recommended. In this study, JRC values for the rock samples obtained from the Tamborine Mountain slop area by two different methods proposed by Barton and Choubey in 1977 and Barton in 1982. By compared with these two methods and the result by converting the Direct shear test data, the JRC values has been determined. This study also includes the damage coefficient for the rock sample from Gold Coast area. Due to the non-homogeneous of the rock, the coefficient for the rock damage is various. The damage coefficient has been conducted into three different method. This study is mainly focused on the method proposed by Barton and Choubey in 1977. In the end, this paper presented the new modulus for the damage coefficient and the JRC values from the calculation is quite accurate compared with the measurement. The shear strength based on the measured JRC values are confirmed with the direct shear test, accurately.

**Keywords:** Rock joint, Shear strength, JRC, damage coefficient.
6.1 Introduction

Failures in jointed rock mass commonly occur in shear along weak discontinuities (or joints). A good deal of research has been conducted to investigate the effect of joint characteristics, including the joint surface roughness, on the shear strength of rock mass. In these studies, the shear strength of jointed rock was estimated using Barton’s criterion (Eq. 6.1) as recommended by ISRM (ISRM, 1978; Sow et al., 2016).

\[
\tau = \sigma_n \tan(\phi_r + JRC \log_{10} \left(\frac{\sigma_{cs}}{\sigma_n}\right))
\]  

where, \(\tau\) is the shear stress, \(\sigma_n\) is the normal stress acting on the joint surface, JCS is the joint wall compressive strength (which is approximately equal to unconfined compressive strength of rock), JRC is the joint roughness coefficient that varies from 0 (smooth, flat surfaces) to 20 (irregular surfaces), and \(\phi_r\) is the residual friction angle. JRC seems to play an important role in the strength of jointed rocks, i.e., irregular surfaces with higher values of JRC tend to produce greater values of shear strength. There are different approaches to obtain JRC, including direct measurements of rock surfaces and estimation of JRC from a series of shear box tests conducted on jointed rock specimens.

Previous research indicates that during shearing, the rock surface can experience damage due to high normal stresses or relatively weak strength of the tested rock. This can change the joint surface characteristics (including JRC) and affect the overall shear strength of jointed rock. To estimate the level of damage that can occur due to high shear forces, a damage coefficient (M) was proposed by Barton (1977), which can be estimated as shown in Eq. 6.2

\[
M = \frac{JRC}{12 \log_{10} \left(\frac{\sigma_{cs}}{\sigma_n}\right)} + 0.7
\]  

This damage coefficient suggests that joints with higher values of JRC (JRC>10) may undergo greater damage. Also, if M is relatively high than greater damage to joint
surface may be expected. Yet, it is still unclear whether Eq.6.2 can accurately estimate the
damage of different rock types as it was experimentally obtained from a series of model
fracture tests, and thus it may significantly vary for natural rocks, depending on rock’s
geological origin.

This study seeks to better understand the effect of joint surface damage on the shear
strength of jointed rocks and clarify the role of damage coefficient in the estimation of rock
strength. A series of shear box tests on two natural jointed rocks were performed and changes
in the joint surface roughness before and after shear were recorded and analyzed. This paper
presents and discusses the obtained results.

6.2 Experiment program

6.2.1 Rock used

Borehole core samples (diameter of 50 mm) of two rock types (namely argillite and
sandstone) were collected from the Gold Coast area (Queensland, Australia) (Figure 6-1).
These rocks were part of the Neranleigh-Fernvale bed formation (Gratchev et al., 2013; Kim
et al., 2013) which are commonly associated with slope stability issues (Kim et al., 2015).
Although the core specimens contained joints, they were identified as fresh to slightly
weathered, following (Kim et al., 2015; Look and Griffiths, 2001). The strength
characteristics and mineral composition of these rocks are summarized in Table 6-1. In this
study, the samples of sandstone are labeled as “S” while the specimens of argillite are
referred to as “B”.

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CHAPTER 6

Figure 6-1 A view of tested specimens (a) sandstone, (b) argillite

Table 6-1 Rock properties

<table>
<thead>
<tr>
<th>Sample Name</th>
<th>Sandstone</th>
<th>Argillite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porosity (%)</td>
<td>16.7</td>
<td>4.9</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>1.99</td>
<td>2.55</td>
</tr>
<tr>
<td>UCS (MPa)</td>
<td>31.2</td>
<td>39.4</td>
</tr>
<tr>
<td>Major minerals</td>
<td>Quartz, Feldspar, Kaolinite</td>
<td>Quartz, Feldspar, Kaolinite, Calcite, Illite, Muscovite</td>
</tr>
<tr>
<td>Mean Iₘ(50) (MPa)</td>
<td>1.9</td>
<td>3.4</td>
</tr>
</tbody>
</table>

6.2.2 Testing Procedure

Experimental program included a series of shear box, tilt, and Barton’s comb tests (Barton and Choubey, 1977).

Shear box tests. Rock specimens with natural and manually created joints were prepared for shear box tests by sizing the core samples to a length of 70-80 mm.

The top and bottom parts of each specimen were secured together with string, an encapsulating mixture of cornice cement (the water – cement ratio was 1:2) was prepared.
and poured into the direct shear test molds in which the sample was partially submerged (Figure 6-2a). When the cement material was strong enough to hold its shape, another pour of cement mixture was placed into a second mold and the other half of the sample was turned upside down and submerged into the new mold. The specimens with the cement cap were then removed from the molds and placed into a dehydrating oven (temperature of 100°C) to cure for about 100 hours. Figure 6-2b presents the specimen setup before a shear box test.

Figure 6-2 Preparation of rock specimens for shear box tests: (a) rock core cast in concrete, and (b) setup before the shear box test

For each test, normal stress was first applied: 1, 3, 5 MPa for the sandstone specimens, and 2, 4, 6, 8 MPa for the argillite specimens. The shear force was then applied in steps to shear the specimen by 0.1 mm until the peak shear stress was achieved.
Barton’s comb measurements. Barton’s comb was used to measure the specimen surface and obtain the corresponding values of JRC before and after shear box tests. The JRC values of these samples varied from as low as 2-4 to as high as 14-16. Table 6-2 summarizes the JRC values of each specimen before and after each test while Figure 6-3 gives an example of changes in the rock surface caused by shear.

![Figure 6-3 A view of rock surface before (a,c) and after (b,d) shear box test. Top and bottom surfaces of S3 (a,b) and A7 (c,d).](image)

Tilt tests. A series of tilt tests were conducted to obtain the basic friction angle of rock specimens with smooth surfaces. The test procedure is described in detail by (USBR; Kim et al., 2016). It was found that an average angle for the sandstone and argillite was 29.8° and 27.8°, respectively.

![Table 6-2 Summary of JRC values measured before and after a shear box test and damage coefficients](image)
<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Estimated JRC</th>
<th>Barton’s Comb Shear box test</th>
<th>JCS/σ\text{n}</th>
<th>Damage coefficient</th>
<th>Modified damage coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>before shear</td>
<td>after shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1</td>
<td>8</td>
<td>8</td>
<td>8.4</td>
<td>20.80</td>
<td>1.21</td>
</tr>
<tr>
<td>S2</td>
<td>8</td>
<td>6</td>
<td>5.7</td>
<td>12.48</td>
<td>1.31</td>
</tr>
<tr>
<td>S3</td>
<td>10</td>
<td>10</td>
<td>11.4</td>
<td>62.40</td>
<td>1.16</td>
</tr>
<tr>
<td>S4</td>
<td>12</td>
<td>8</td>
<td>13.0</td>
<td>20.80</td>
<td>1.46</td>
</tr>
<tr>
<td>S5</td>
<td>14</td>
<td>12</td>
<td>11.4</td>
<td>20.80</td>
<td>1.59</td>
</tr>
<tr>
<td>S6</td>
<td>14</td>
<td>14</td>
<td>8.5</td>
<td>12.48</td>
<td>1.76</td>
</tr>
<tr>
<td>S7</td>
<td>14</td>
<td>14</td>
<td>15.7</td>
<td>62.40</td>
<td>1.35</td>
</tr>
<tr>
<td>A1</td>
<td>5</td>
<td>2</td>
<td>6.3</td>
<td>9.85</td>
<td>1.12</td>
</tr>
<tr>
<td>A2</td>
<td>6</td>
<td>4</td>
<td>7.6</td>
<td>13.13</td>
<td>1.15</td>
</tr>
<tr>
<td>A3</td>
<td>4</td>
<td>4</td>
<td>10.7</td>
<td>13.13</td>
<td>1.00</td>
</tr>
<tr>
<td>A4</td>
<td>8</td>
<td>8</td>
<td>9.1</td>
<td>15.76</td>
<td>1.26</td>
</tr>
<tr>
<td>A5</td>
<td>12</td>
<td>8</td>
<td>12.4</td>
<td>15.76</td>
<td>1.54</td>
</tr>
<tr>
<td>A6</td>
<td>16</td>
<td>12</td>
<td>16.2</td>
<td>13.13</td>
<td>1.89</td>
</tr>
<tr>
<td>A7</td>
<td>16</td>
<td>14</td>
<td>16.8</td>
<td>13.13</td>
<td>1.89</td>
</tr>
</tbody>
</table>

### 6.3 Test result and discussion

#### 6.3.1 Shear box test

Typical results of shear box tests are given in Figure 6-4 showed the effect of JRC on the shear strength of jointed rock. For sandstone (Figure 6-4a), the specimens with higher values of JRC tend to produce greater values of peak shear strength. The same tendency is observed for argillite (Figure 6-4b); that is, the specimens with JRC≈10 have slightly greater...
values of shear strength.

All tests results were replotted in Figure 6–5 to better demonstrate the effect of surface roughness on shear strength. Regardless of rock type, the shear strength of jointed rock tends to increase with increasing values of JRC.

Figure 6-4 Typical shear box test results obtained for a) sandstone, and b) argillite
Figure 6-5 Summary of shear box test results plotted as peak shear stress at corresponding values of normal stress: a) sandstone, and b) argillite

6.3.2 Changes in joint roughness after shearing

Table 6-2 summarizes laboratory data of JRC values measured before and after the shear box tests as well as the JRC values estimated using Eq.6.3, where values of $\sigma_n$ and $\tau$ were obtained from a series of shear box tests.
To better understand the effect of shear on joint surface roughness, the laboratory data was re-plotted in Figure 6-6 as JRC measured by Barton’s comb before and after the shear box test and JRC obtained from the shear box test using Eq. 6.3. It is evident from this figure that for both rocks, the JRC value tends to decrease after the shearing process. It can be attributed to the breakage or smoothing of surface irregularities caused by normal and shear forces acting on the joint surface. It is interesting to note that changes in JRC of argillite also occurs for relatively smooth surfaces (low JRC values). In contrast, Eq 6.2 tends to overestimate the JRC for both rocks, which is more pronounced for argillite.

Figure 6-6 Summary of data on JRC values before and after shear: a) sandstone, and b) argillite
6.3.3 The damage coefficient t

To estimate the damage potential of joint surfaces during shearing, the coefficient of damage (M) was obtained for each test using Eq.6.2 (Table 6-2). It is evident that for each case, M was greater than 1, suggesting considerable damage to the surface. However, visual observations and direct measurements performed after each test indicate that in some cases, the damage was rather insignificant. This discrepancy suggests that Eq.6.2 may not be suitable for all rock types, and care needs to be taken when applying this coefficient for a particular type of rock.

Considering the obtained results, the new mathematical expressions of the damage coefficient for sandstone (Eq.6.4) and argillite (Eq.6.5) were proposed. New values of the modified damage coefficient are given in Table 2.

$$M = \frac{J_{RC}}{12 \cdot \log_{10} \left( \frac{J_{CS}}{\sigma_n} \right)} + 0.4 \quad 6.4$$

$$M = \frac{J_{RC}}{12 \cdot \log_{10} \left( \frac{J_{CS}}{\sigma_n} \right)} + 0.6 \quad 6.5$$

The modified damage coefficient (M) better correlates with the changes in JRC values, i.e., no or little changes occur when M<0 while some to considerable changes can be expected when M>1.

Figure 6-7 plots that obtained results as the modified damage coefficient against the ratio of JCS/σn. According to Baron and Choubey (1977), the damage of joint surface may occur when the ratio is relatively low. The results obtained in this study seem to be in agreement with this statement as changes in JRC occurred when the JCS/σn was less than 20.
6.4 Conclusions

A series of shear box tests on rock specimens of jointed sandstone and argillite were performed to estimate the effect of joint surface damage on overall shear strength. Based on the obtained results, the following conclusions can be drawn:

- The shear strength of the tested rocks depends on the joint roughness; i.e., it increases when the joint surface becomes more irregular.

- Damage to joint surface can occur during shear, thus affecting the shear strength of rock. It was found that the damage may even occur to relatively flat, smooth surfaces.
(JRC<10), and not only to highly irregular surfaces (JRC>10), as was reported by other researchers.

- The damage coefficient (M) can be used to estimate the degree of damage that can occur during shearing of jointed rocks. When M<1, no or little damage may be expected during shearing while considerable damage to the joint surface may occur when M>1. However, it was found that M depends on the type of rock, and it varies with the rock geology.
Chapter 7

THE EFFECT OF CLAY INFILL ON STRENGTH OF JOINTED SANDSTONE: LABORATORY AND NUMERICAL ANALYSIS

Statement of contribution to co-authored published paper

This chapter includes a co-authored paper. The bibliographic details of the co-authored paper, including all authors, are:


Note that the format of this paper has been changed according to the guidelines of the thesis, and supplementary experimental details which were not included in this paper for publication now have been provided in “APPENDIX 2” for references. My contribution to the paper involved: literature review, experimental data analysis, discussion of test results and development of analytical study, paper writing and responding reviewers.

(Signed) ___________________________ (Date) __15/12/2022____________
Chen Cui

(Countersigned) ___________________________ (Date) __15/12/2022____________
Principal Supervisor: A/P Gratchev Ivan
CHAPTER 7: THE EFFECT OF CLAY INFILL ON STRENGTH OF JOINTED SANDSTONE: LABORATORY AND NUMERICAL ANALYSIS

Abstract

The strength of jointed rock is a fundamental factor in the slope stability of rock mass. This aim of this research is to investigate the effect of infill thickness on the strength of jointed rock specimens. Unlike previous studies which involved artificial rock-like materials and saw-tooth surfaces, this work has been conducted on two natural types of sandstone having various rock surfaces. Natural low-plasticity clay of different thickness (ranging from 1 mm to 3 mm) was used as the infill material. A series of shear box tests with a range of initial normal stresses from 0.5MPa to 1.5MPa was performed to obtain high-quality data regarding the shear strength of natural rock and provide insights into the effect of infill and rock surface roughness on shear strength. The obtained results were also used to improve the current methods of rock strength predictions, which were initially designed to estimate the strength of artificial rock-like material. The newly proposed procedure proved to provide more accurate estimations of the shear strength of jointed rock.

Highlights

- Unlike most of previous studies, this research is related to natural rocks (sandstone) with natural infills.
- Natural rock surfaces have been used instead of the saw-tooth model.
• The newly proposed numerical model provides more accurate prediction of shear strength of joined rock.

**Keywords:** Natural rock joint; Natural infill material; Rough surface; Numerical prediction.
7.1 Introduction

Discontinuities in rock mass, including joints, bedding planes, and faults, can significantly affect the rock shear strength, resulting in slope failures (Patton, 1966; Barton, 1973; Hoek and Brown, 1980; Gratchev et al., 2013; Raghuvanshi, 2019; Ram and Basu, 2019; Saadat and Taheri, 2019; Cui and Gratchev, 2020; Xia et al., 2022). In the field, the rock joints are typically filled with natural material such as soil. To study the effect of infill material on the strength of jointed rock mass, a series of laboratory studies under constant normal load (CNL) condition has been conducted (Kanji, 1974; Phien-Wej et al., 1990; Jahanian and Sadaghiani, 2015; Cheng et al., 2016; Lu et al. 2017; Kang et al., 2019; Wu et al., 2019; Zhao et al., 2020; Zhao et al., 2021; Berisavljevic et al., 2022; Hu et al., 2022; Kasyap and Senetakis, 2022; Zhao et al., 2022). In addition, research into constant normal stiffness (CNS) condition (Indraratna et al., 1999, 2005; Indraratna and Welidniya, 2003; Brown, 2004; Oliveira et al., 2009) has been carried out (see Table 7-1).

These studies showed the shear behaviour of the infilled joints depends on the surface roughness, type of infill material, and the ratio of the infill thickness (t) to the asperity height (a), i.e., t/a (Phien-Wej et al., 1990; Indraratna et al., 2005, 2008; Oliveira et al., 2009; Naghadehi, 2015). This research has showed that when the infill thickness increases, the shear strength of the jointed rock mass tends to decrease.

The t/a ratio seems to have a strong influence on the shear strength of rock. To investigate this, several studies have been conducted using artificial rock joint models as shown in Figure 7-1a. It was found that when the t/a ratio is equal to or less than 1, shearing mostly occurs between the joints and the infill material (Indraratna and Mylvaganam, 2005). When the t/a ratio ranges from 0.35 to 0.72, the shear strength of the jointed rock may decrease (Lama, 1978); and it can significantly drop when the t/a ratio becomes less than
For relatively large values of t/a ratio (more than 1.25), according to Goodman (1970) and De Toledo et al., (1993), shear may also occur inside the infill material. Jahanian and Sadaghiani (2015) noted that when the t/a ratio is greater than 2, the shear would occur in the infill material only.

Table 7-1 Summary of the previous research about the t/a ratio and the infill material

<table>
<thead>
<tr>
<th>Type of model</th>
<th>Material</th>
<th>Thickness over asperity (t/a)</th>
<th>Infill material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Papaliangas et al. (1993)</td>
<td>Rough surface</td>
<td>Plaster</td>
<td>1, 1.5, 1.14</td>
</tr>
<tr>
<td>Indraratna et al. (2005)</td>
<td>Saw-tooth model</td>
<td>Gypsum plaster</td>
<td>0.6, 1.2, 1.8, 3.6</td>
</tr>
<tr>
<td>Indraratna and Mylvaganam (2005)</td>
<td>Saw-tooth model</td>
<td>Gypsum plaster</td>
<td>0, 1, 2, 3.5</td>
</tr>
<tr>
<td>Indraratna et al. (2008)</td>
<td>Saw-tooth model</td>
<td>Gypsum plaster</td>
<td>0.5, 1, 1.5, 2, 3.5</td>
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<td>Oliveira et al. (2008)</td>
<td>Saw-tooth model</td>
<td>Gypsum plaster</td>
<td>0.5, 1, 1.5, 2</td>
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<td>Indraratna et al. (2014)</td>
<td>Saw-tooth model</td>
<td>Gypsum plaster</td>
<td>0.26, 0.51, 1.53, 2.05</td>
</tr>
<tr>
<td>Naghadehi (2015)</td>
<td>Saw-tooth model</td>
<td>Sandstone</td>
<td>0, 0.2, 0.4, 0.6, 0.8, 1, 1.2, 1.4, 1.6</td>
</tr>
<tr>
<td>Jahanian and Sadaghiani (2015)</td>
<td>Saw-tooth model</td>
<td>Gypsum plaster</td>
<td>0.3, 0.5, 0.6, 1, 1.2, 2</td>
</tr>
</tbody>
</table>
CHAPTER 7

<p>| | | | | |</p>
<table>
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<tr>
<th></th>
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<tr>
<td>Liu et al.</td>
<td>Saw-tooth</td>
<td>Sandstone</td>
<td>0, 0.05, 0.1,</td>
<td>cement</td>
</tr>
<tr>
<td>(2017)</td>
<td>model</td>
<td></td>
<td>0.25, 0.25, 0.5,</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>0.75, 1, 1.5, 2</td>
<td></td>
</tr>
<tr>
<td>Zhao et al.</td>
<td>Rough surface</td>
<td>Cement</td>
<td>0.614, 0.725,</td>
<td>Sand and clay</td>
</tr>
<tr>
<td>(2020)</td>
<td></td>
<td></td>
<td>1.03, 1.89</td>
<td></td>
</tr>
<tr>
<td>Li et al.</td>
<td>Saw-tooth</td>
<td>Sandstone,</td>
<td>1 mm(infill)</td>
<td>cement</td>
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<tr>
<td>(2022)</td>
<td>model</td>
<td>mudstone and cement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>This study</td>
<td>Rough surface</td>
<td>Rock core</td>
<td>1mm, 2mm, 3mm (infill)</td>
<td>clay</td>
</tr>
<tr>
<td>(2022)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Although the aforementioned studies have provided useful insight into the shear behaviour of joints filled with various materials, they also have some shortcomings:

Firstly, as shown in Table 7-1, most studies have employed artificial saw-tooth models, while only limited research has been performed on irregular surfaces of natural rock. However, such irregular surfaces are encountered in the field. Figure 7-1 shows the difference between the artificially created rock surfaces (Figure 7-1a) and the surface of natural rock (Figure 7-1b).

Secondly, in experimental studies, most researchers have used rock-like material, such as plaster and cement, which may not accurately represent the natural rock conditions. Only limited studies have been conducted on natural rocks. These have indicated the importance of a lack of rock inhomogeneity on the engineering properties.
Figure 7-1 Cross-section (a) saw-tooth model with infill, (b) natural core rock joint

On the basis of this knowledge gap, this study seeks to investigate the shear behaviour of natural rocks with and without the infill material, to improve our understanding of the major factors that determine the strength of natural rock. A series of shear box tests was carried out on two types of sandstone with irregular rock surfaces. To estimate the effect of infill thickness on shear strength, moist clay was used as the infill. The obtained data were used to refine the existing numerical methods, in order to estimate the effect of infill on the shear strength of jointed sandstone. The newly proposed method has shown more accurate estimations of the shear strength, compared to the previous methods which were mainly designed based on artificial rock-like material.

7.2 Theoretical considerations on the shear strength of jointed rock

To estimate the effect of infill on shear the strength of jointed rock, a few numerical studies have been conducted (Barton and Choubey, 1977; Lama, 1978; Papaliangas et al., 1993; Pereira, 1997; Indraratna et al., 1999; Sinha and Singh., 2000; Indraratna et al., 2008; Indraratna et al., 2010; Xu et al., 2013; Indraratna et al., 2014; Jahanian and Sadaghiani,
For jointed rock with no infill, Barton’s equation (Eq 7.1) has been widely used.

\[
\tau = \sigma_n \tan (\phi_r + JRC \log \left( \frac{JCS}{\sigma_n} \right))
\]  

(7.1)

where, \(\sigma_n\) is the normal stress on the joint, \(\phi_r\) is the residual friction angle, and the joint wall compressive strength (JCS) is, equal to the unconfined compressive strength (UCS).

Indraratna et al. (2008), have combined the normalized shear strength \(\left( \frac{\tau_p}{\sigma_n} \right)_{oc,n} \) with the infill thickness over the asperity height \((t/a)\) to obtain Eqs 7.2 to 7.4:

\[
\left( \frac{\tau_p}{\sigma_n} \right)_{oc,n} = A_n + B_n
\]  

(7.2)

\[
A_n = \tan (\phi_b + i_0) \times (1 - K_{oc,n})^{a_n}
\]  

(7.3)

\[
B_n = \tan \varphi_{fill} \times OCR^\alpha \times \left( \frac{2}{1 + k_s} \right)^{b_n}
\]  

(7.4)

where, \(K_{oc,n}\) is the factor of the \(t/a\) ratio of the testing sample over the critical \(t/a\) ratio. Also in the present study, the interfering zone was estimated as proposed by Indraratna et al. (2010). \(i_0\) is the initial asperity angle and \(a_n\), \(b_n\) are the empirical constants. Indraratna has taken the consolidation ratio into consideration with an idealised joint roughness. Indraratna et al. (2013) modified these equations correlated with the degree of saturation of the infilled material. The algebraic function \(\left( \frac{\tau_p}{\sigma_n} \right)\) proposed by Indraratna et al. (2005), was applied (Eqs 7.5 and 7.6),

\[
A = \left[ \sigma_n \times \tan (\phi_b + JRC \times \log \left( \frac{JCS}{\sigma_n} \right) \right] \times (1 - k_s)^\alpha
\]  

(7.5)

\[
B = C_t + \left[ (\sigma_n - u_a) \times \tan \varphi' \right] \times \left( \frac{2}{1 + k_s} \right)^\beta
\]  

(7.6)

\[
\tau = \left[ \sigma_n \times \tan (\phi_b + JRC \times \log \left( \frac{JCS}{\sigma_n} \right) \right] \times (1 - k_s)^\alpha + C_t + \left[ (\sigma_n - u_a) \times \tan \varphi' \right] \times \left( \frac{2}{1 + k_s} \right)^\beta (k_s = \frac{(t/a)}{(t/a)_{cr}} < 1)
\]  

(7.7)
\[ \tau = c_t + (\sigma_n - u_a)\tan\phi' \]  \hspace{1cm} (7.8)

The boundary of the interfering zone is based on the ratio of the critical \( t/a \) ratio and the given \( t/a \) ratio both at same consolidation rate. As for the interfering and non-interfering conditions, Eq 7.7 applies only when \( k_s \) is less than 1, which means it is under interfering zone. When \( k_s \) is greater than 1, the shear strength is calculated by Eq 7.8.

Note that in Eqs. 7.7 and 7.8, \( u_a \) is the pore air pressure which is normally taken to be the normal air pressure. Thus, for a relatively thicker infilled joint, the shear mostly happens in the filling material instead of both the rock joint and the infill.

Lama (1978) proposed a method that only considered the infill thickness, without the separately taking into account the asperity height or the joint roughness (Eq 7.9).

\[ T_p = 7.25 + 0.46 \times \sigma_n - 0.3 \times \ln(f) \times \sigma_n^{0.745} \]  \hspace{1cm} (7.9)

where \( T_p \) is the shear strength (MPa), \( \sigma_n \) is the normal strength (MPa), and \( f \) is the thickness of infill material.

Papaliangas et al. (1993) and Jahanian and Sadaghiani (2015), made some modifications that linked the shear strength with the \( t/a \) ratio, which has been proposed as Eqs 7.10 to 7.13:

\[ \mu = \mu_{\text{min}} + (\mu_{\text{max}} - \mu_{\text{min}})^n \]  \hspace{1cm} (7.10)

\[ \mu_{\text{min}} = \frac{T_{\text{min}}}{\sigma} \times 100 \]  \hspace{1cm} (7.11)

\[ \mu_{\text{max}} = \frac{T_{\text{max}}}{\sigma} \times 100 \]  \hspace{1cm} (7.12)

\[ n = \left(1 - \frac{1}{c} \times \frac{t}{a}\right)^m \]  \hspace{1cm} (7.13)

Where, \( \mu \) is the shear strength, \( c \) and \( m \) are the experimentally derived constant, \( T_{\text{max}} \) is the maximum shear strength of unfilled joints and \( T_{\text{min}} \) is the potential minimum shear strength of the system for the critical thickness of the infill. In order to minimize the use of empirical constants, Zhao et al. (2020) proposed incorporating the friction angle of the infilled joint...
and the cohesion of the infill material, thus modifying Barton and Choubey’s theory to obtain a new method (Eqs 7.14 and 7.15):

$$\tau_p = \sigma_n \times \tan \left\{ \left[ JRC \times \log \left( \frac{\sigma_n}{c_n} \right) + \phi_p \right] \times \exp \left( -\beta \times \frac{t}{a} \right) \right\} + c_{\text{infill}} \quad (7.14)$$

$$\tau_p = \sigma_n \times \tan \phi_{\text{infill}} + c_{\text{infill}} \quad (7.15)$$

In Eqs. 7.14 and 7.15, the friction angle of the rock joint has been modified by using the original joint with a new function which related to the t/a ratio. The coefficient in the new friction angle, which is the infilled joint friction angle $\beta$, is a decay coefficient. $\beta$ is a correlation to the friction angle of the infilled material and the joint roughness coefficient (JRC) value, which was presented in the studies by Papaliangas et al. (1990, 1993), Naghadehi (2015), Jahanian and Sadaghiani (2015), and Zhao et al. (2020). As with Indraratna’s approach, there still is a boundary condition of the interfering and the non-interfering zones, and the t/a ratio is compared with the critical t/a ratio.

From the above-mentioned studies, it can be concluded that most previous research has been converging on use of the saw-tooth model and rock like material. The influences of natural rock and a rough surface have been inadequately investigated. The aim of the present study is to examine the effect of infill with different thicknesses on natural rock with a rough surface. The differences in numerical calculations based on these two factors were investigated and the procedure for the analysis was also refined.

### 7.3 Experimental program

#### 7.3.1 Joint sample and infill sample

Two types of sandstone were collected from the Gold Coast area, Australia. They were part of the Marburg Formation and Neranleigh-Fernvale Beds, respectively (Gratchev et al.,
2013; Gratchev and Kim, 2016; Gratchev et al., 2019). Sandstone 1 (S1) had a light-yellow colour, and it was relatively soft. Sandstone 2 (S2) was of light-grey colour, with layers of fine material (Figure 7-2).

![Image of rock sample under photomicroscope](image)

**Figure 7-2** Rock sample under photomicroscope: (a) sandstone 1, (b) sandstone 2.

Figure 7-2 shows the cross-section of these two rock samples: S1 had a greater particle size range, from 600 μm to 3000 μm, while S2’s range is only 300 μm to 600 μm. Figure 7-3 presents the mineral composition of both rocks. S1 contains quartz, kaolinite and feldspar, while S2 was made of quartz, calcite, kaolinite. S1 had a relatively high porosity of 16.7%, a lower density of 1.99 g/cm³ and UCS for 18.7 MPa. Sandstone 2 had a porosity of 8.4%, a density of 2.57 g/cm³ and UCS for 47.5 MPa.
Figure 7-3 XRD for two sandstones: (a) sandstone 1, (b) sandstone 2.)
Infill clay (classified as CL) had a dry density of 1.8 g/cm$^3$ and a moisture content of 25%. Figure 7-4 presents the summary of shear box tests on this clay, giving the friction angle of 27.3° and cohesion of 17.4 kPa.

![Figure 7-4 Result of shear box test on the infilled clay](image)

7.3.2 Experimental program

Barton’s comb was used to measure the rock surface roughness. Tilt test was used to estimate the residual friction angle of the rock (Barton and Choubey, 1977). To estimate the strength of the rock, a series of unconfined compression tests were conducted on cylindrical samples (diameter of 50 mm and height of 100 mm) following the Australia standard AS 1141-1996 (1996). For the direct shear test, a portable shear box SL 900 was used, with a sample diameter of 50 mm. A range of normal stresses (from 0.5 MPa to 1.5MPa) was used. Strain-controlled shear box tests were conducted on several rock specimens with and without the clay infill. Three different infill thicknesses, 1mm, 2mm, and 3mm, were used for both sandstones.
CHAPTER 7

7.4 Results and discussion

7.4.1 Shear strength of joints without infill.

Table 7-2 shows the results of the direct shear test and the estimated shear stress by using the Barton and Choubey (1977) method. As Table 7-2 indicates, the shear strength of S2 is greater than S1 under various kinds of normal stress which is same as the UCS result.

<table>
<thead>
<tr>
<th>Sample number</th>
<th>JRC</th>
<th>Normal Stress (MPa)</th>
<th>Peak Shear Stress (MPa)</th>
<th>Estimated Shear stress (MPa)[eq.1]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A1</td>
<td>17</td>
<td>0.25</td>
<td>0.72</td>
<td>0.52</td>
</tr>
<tr>
<td>A2</td>
<td>18</td>
<td>0.5</td>
<td>1.09</td>
<td>0.89</td>
</tr>
<tr>
<td>A3</td>
<td>15</td>
<td>0.75</td>
<td>1.13</td>
<td>1.01</td>
</tr>
<tr>
<td>A4</td>
<td>12</td>
<td>1</td>
<td>1.18</td>
<td>1.01</td>
</tr>
<tr>
<td>A5</td>
<td>12</td>
<td>1.25</td>
<td>1.01</td>
<td>1.31</td>
</tr>
<tr>
<td>A6</td>
<td>10</td>
<td>1.75</td>
<td>2.10</td>
<td>2.01</td>
</tr>
<tr>
<td>A7</td>
<td>16</td>
<td>2</td>
<td>1.99</td>
<td>1.80</td>
</tr>
<tr>
<td>Sandstone 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B1</td>
<td>17</td>
<td>0.25</td>
<td>0.72</td>
<td>0.71</td>
</tr>
<tr>
<td>B2</td>
<td>18</td>
<td>0.5</td>
<td>1.09</td>
<td>1.21</td>
</tr>
<tr>
<td>B3</td>
<td>15</td>
<td>0.75</td>
<td>1.13</td>
<td>1.25</td>
</tr>
<tr>
<td>B4</td>
<td>14</td>
<td>1</td>
<td>1.55</td>
<td>1.45</td>
</tr>
<tr>
<td>B6</td>
<td>13</td>
<td>1.5</td>
<td>1.33</td>
<td>1.88</td>
</tr>
<tr>
<td>B7</td>
<td>13</td>
<td>1.75</td>
<td>2.10</td>
<td>2.13</td>
</tr>
<tr>
<td>B8</td>
<td>15</td>
<td>2</td>
<td>2.80</td>
<td>2.62</td>
</tr>
</tbody>
</table>

Figures 7-5 and 7-6 show the direct shear test results for S1 and S2 with different infill thicknesses.
Figure 7-5 S1 shear strength vs displacement with different thicknesses of infill (a)

\[ \sigma_n = 0.5 \, MPa, \quad (b) \quad \sigma_n = 1 \, MPa \]
Figure 7-6 S2 shear strength vs displacement with different thicknesses of infill (a)

\[ \sigma_n = 1 \text{ MPa}, (b) \sigma_n = 1.5 \text{ MPa} \]

It is evident from Figures 7-5 and 7-6 that for both S1 and S2, the shear strength of the jointed specimens was significantly greater without the infill material expect for the normal stress 0.5MPa. When the clay infill was used, the shear strength decreased by twofold, which
can be attributed to the effect of the softer clay. This finding indicates that the presence of clay on the rock surface tends to decrease the rock’s strength. It is interesting that the opposite effect was observed for the same rock with a greater normal stress of 1 MPa. As shown in Figures 7-5b and 7-6b, the shear strength of the jointed rock specimens without the infill material was lower for both sandstones.
Figure 7-7 Shear strength vs different infill thickness: (a) sandstone 1, (b) sandstone 2

To better understand the effect of infill and the initial normal stress, the laboratory data was replotted in Figure 7-7 as the peak shear strength against the infill thickness. For S1 under the normal stress of 0.5 MPa, the shear strength of the infilled joints was observed to be relatively low compared to the specimens with no infill. Also, an increase in the infill thickness had an opposite effect on shear strength. For example, for S1, under a relatively low normal stress of 0.5MPa, the shear strength tended to slightly increase with the infill thickness increase from 1mm to 3mm. However, under a greater normal stress of 1 MPa, the shear strength tended to slightly decrease as the infill thickness increased.

Visual observations demonstrated that there may be three major factors that affect the shear strength of the jointed rock with infill. Firstly, for the range of the infill thicknesses tested in this study, an increase in the infill thickness tends to slightly decrease the shear strength of jointed rock. Secondly, the presence of 1mm infill can slightly increase the shear strength of the jointed rock. Finally, rock inhomogeneity and surface roughness can contribute to the shear strength.

7.4.2 Numerical analysis of the obtained results

Numerical analysis of the obtained laboratory data was conducted using the methods proposed by Indraratna et al.(2014) (Eq. 7) and Zhao et al.(2020) (Eq. 14), the results of which, are summarized in Table 3.

Table 7-3 Summary of the numerical analysis using different methods
<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Normal stress (MPa)</th>
<th>Infill thickness (mm)</th>
<th>t/a</th>
<th>Estimated shear strength [Eq.7] (MPa)</th>
<th>Estimated shear strength [Eq.14] (MPa)</th>
<th>Experiment tested shear strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A8</td>
<td>0.5</td>
<td>1</td>
<td>1.00</td>
<td>0.46</td>
<td>0.34</td>
<td>0.64</td>
</tr>
<tr>
<td>A9</td>
<td></td>
<td>1</td>
<td>0.36</td>
<td>0.31</td>
<td>0.47</td>
<td>0.70</td>
</tr>
<tr>
<td>A10</td>
<td></td>
<td>2</td>
<td>0.57</td>
<td>0.57</td>
<td>0.38</td>
<td>0.77</td>
</tr>
<tr>
<td>A11</td>
<td></td>
<td>2</td>
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<td>0.54</td>
<td>0.38</td>
<td>0.70</td>
</tr>
<tr>
<td>A12</td>
<td></td>
<td>3</td>
<td>0.67</td>
<td>0.61</td>
<td>0.28</td>
<td>0.82</td>
</tr>
<tr>
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<td></td>
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<td>1.50</td>
<td>0.62</td>
<td>0.27</td>
<td>0.83</td>
</tr>
<tr>
<td>A14</td>
<td>1.0</td>
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<td>0.50</td>
<td>1.01</td>
<td>0.71</td>
<td>1.342</td>
</tr>
<tr>
<td>A15</td>
<td></td>
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<td>0.96</td>
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<td>0.858</td>
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<td>0.72</td>
<td>1.295</td>
</tr>
<tr>
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<td>2</td>
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<td>0.76</td>
<td>0.52</td>
<td>1.295</td>
</tr>
<tr>
<td>A18</td>
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<td>3</td>
<td>1.50</td>
<td>0.76</td>
<td>0.40</td>
<td>1.295</td>
</tr>
<tr>
<td>A19</td>
<td></td>
<td>3</td>
<td>1.50</td>
<td>0.75</td>
<td>0.48</td>
<td>1.295</td>
</tr>
<tr>
<td>B9</td>
<td>1.0</td>
<td>1</td>
<td>0.29</td>
<td>0.91</td>
<td>1.06</td>
<td>1.490</td>
</tr>
<tr>
<td>B10</td>
<td></td>
<td>1</td>
<td>0.35</td>
<td>0.84</td>
<td>1.01</td>
<td>1.597</td>
</tr>
<tr>
<td>B11</td>
<td></td>
<td>2</td>
<td>0.59</td>
<td>0.61</td>
<td>0.84</td>
<td>1.447</td>
</tr>
<tr>
<td>B12</td>
<td></td>
<td>2</td>
<td>1.19</td>
<td>0.48</td>
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</tr>
<tr>
<td>B13</td>
<td></td>
<td>3</td>
<td>1.06</td>
<td>0.45</td>
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<td>0.767</td>
</tr>
<tr>
<td>B14</td>
<td></td>
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<td>1.26</td>
<td>0.50</td>
<td>0.62</td>
<td>0.767</td>
</tr>
<tr>
<td>B15</td>
<td></td>
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<td>0.28</td>
<td>1.36</td>
<td>1.51</td>
<td>1.597</td>
</tr>
<tr>
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<td></td>
<td>1</td>
<td>0.33</td>
<td>1.21</td>
<td>1.44</td>
<td>1.086</td>
</tr>
<tr>
<td>B17</td>
<td></td>
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</tr>
<tr>
<td>B18</td>
<td></td>
<td>2</td>
<td>1.54</td>
<td>1.02</td>
<td>0.78</td>
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</tr>
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<td>0.71</td>
<td>0.98</td>
<td>1.517</td>
</tr>
<tr>
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<td></td>
<td>3</td>
<td>0.86</td>
<td>0.71</td>
<td>0.98</td>
<td>1.977</td>
</tr>
</tbody>
</table>

The difference observed between the laboratory data and the estimated values using Eqs. 7.7 and 7.14 can be attributed to the fact that Indraratna et al. (2014) and Zhao et al. (2020) employed a plaster gypsum material with a tooth-saw surface while the current study used real rock with irregular surfaces. Figure 7-8 summarizes the test data and prediction results for S1 and S2.
It is evident from this figure that the existing methods tend to underpredict the shear strength of natural rocks with infill for both sandstones. The results presented in Table 7-3 suggest that the discrepancy between the measures and predicted results increases with increasing infill thickness. It seems that these methods, although considering the infill cohesion, do not include the friction component of the infill in the shear strength analysis. This is thought to be a reasonable assumption for the saw-tooth model or smooth joints, where the inter-friction between the infill material and the joint surface can be neglected.
Figure 7-8 Experiment data and the theoretical results: (a) S1, (b) S2

However, for irregular rock surfaces like the ones used in this study, there may be a greater effect of the friction between the rock surface and infill, especially when the thickness of the infill material increases (Naghadehi, 2015). It appears that under relatively
large normal stresses, friction also occurs between the rock surface and the infill material. Considering this, the existing methods can be refined by including the friction component as shown in Eqs 7.16 and 7.17.

\[
\tau' = \sigma_n \tan \varphi' + c'
\]  
\[
\tau_p = \sigma_n \times \tan \left\{ [JRC \times \log \left(\frac{JCS}{\sigma_n}\right) + \varphi_p] \times \exp \left(-\beta \times \frac{t}{a}\right) \right\} + \tau'
\]

where, \( \varphi' \) is the friction angle between the joint surface and the infill material, \( c' \) is the cohesion of the infill material.

Figure 7-9 presents a comparison of the laboratory results and the estimated data using the newly proposed Eq. 17. It is clear that Eq. 17 produces more accurate estimates of the shear strength of both sandstones with a different infill thickness. The numerical analysis suggests that when the t/a ratio is less than 1.2 (for S1) and 1.5 (for S2), more accurate predictions can be obtained. It is noted that, unlike the artificially made rock-like specimens, natural rocks may contain void and defects that are not visible during testing but still affect their overall strength. This should be also considered when Eq. 17 is used to estimate the shear strength of rock.
Figure 7-9 Tested result and the modified result (a) S1 (b) S2

7.5 Concluding remarks

A series of shear box tests on natural samples of sandstone has been conducted to study the effect of clay infill on the shear strength of natural rock. Numerical methods were
employed to estimate the shear strength of infilled rock specimens. Based on the obtained results, the following conclusions can be drawn:

- Although the effect of the infill is commonly associated with the decrease of the overall strength of jointed rock, the obtained data indicated that an increase in infill thickness from 1mm to 3 mm can slightly increase the shear strength of natural rock. This can be attributed to the rock inhomogeneity which is not possible to observe in the rock-like material.

- The critical ratio of t/a is related to the type of rock. In this study, the critical ratio obtained for S1 and S2 is relatively lower (1.2 and 1.5, respectively) compared to the one proposed by other researchers who used artificial rock-like material such as gypsum plaster and/or cement. The t/a ratio may vary due to the irregular shape of the joint surface, which highlights the limitation of the commonly used saw-tooth models.

- The existing methods of shear strength prediction may not accurately estimate the shear strength of rock specimens with irregular surfaces. The newly proposed method takes into account the interaction between the rock surfaces and the infill material, which provides a more accurate estimation of the strength.
Chapter 8

NUMERICAL ANALYSIS OF THE EFFECT OF PRE-EXISTING CRACKS WITH INFILL ON THE STRENGTH OF NATURAL ROCKS

Statement of contribution to co-authored published paper

This chapter includes a co-authored paper. The bibliographic details of the co-authored paper, including all authors, are:


Note that the format of this paper has been changed according to the guidelines of the thesis. My contribution to the paper involved: literature review, numerical simulation, discussion of test results and development of analytical study, paper writing and responding reviewers.

(Signed) _________________________________ (Date) __15/12/2022__________________
Chen Cui

(Countersigned) ___________________________ (Date) __15/12/2022__________________
Principal Supervisor: A/P Gratchev Ivan
CHAPTER 8: NUMERICAL ANALYSIS OF THE EFFECT OF PRE-EXISTING CRACKS WITH INFILL ON THE STRENGTH OF NATURAL ROCKS

Abstract

In geotechnical engineering, the strength of jointed rock always associates with different flaws or discontinuities. This study proposes a numerical model that can accurately estimate the strength of jointed rock using a universal distinct element code (UDEC) program. Four natural rocks (two types of sandstone, argillite, and basalt) with pre-existing discontinuities of either 1 cm or 2 cm long filled with sand or clay were tested in unconfined compression, and the obtained data was compared with the numerical results. It was found that the newly developed model estimated the stress-strain behavior and unconfined compressive strength of rocks to a high level of accuracy, and the failure patterns given by the numerical model were in agreement with the data obtained in the laboratory. However, some discrepancies were observed for the argillite and basalt specimens that exhibited anisotropic properties which are common for natural rocks but still rather difficult to incorporate in numerical analysis.

Highlights:

• Unlike most of previous studies, this research is related to natural rocks (sandstone, argillite, and basalt) with natural infills (sand and clay).
• The numerical model was developed by UDEC, and the results of numerical simulation were verified.
• The strain-stress distributions and failure patterns were studied through a new numerical simulation method.

**Key words:** Numerical model; natural rock; pre-existing discontinuities; natural infill; strength behavior.
8.1 Introduction

Discontinuities can significantly affect the rock mass strength and its overall stability (Eberhard et al., 1998; Cui et al., 2019; Kim et al., 2019). A good deal of research has been conducted to investigate the effect of pre-existing cracks on rock unconfined compressive strength (UCS) and shear strength (Bobet and Einstein, 1998; Indraratna et al., 2014; Kim et al., 2014; Cao et al., 2015; Gratchev et al., 2016; Zhou et al., 2018; Zhu et al., 2019; Kasyap and Senetakis, 2022; Zhao et al., 2022). These studies, which mainly involved rock-like material with different crack geometries, showed that the process of crack initiation and propagation were related to the characteristics of the existing cracks and the properties of the rock bridges between those cracks. However, only limited research has been carried out on natural rocks with different geological origins (Wong et al., 2009; Cui and Gratchev, 2020; Li et al., 2022). These studies indicated that the inhomogeneity can greatly contribute to the strength of natural rock. In addition, the influence of infill material, which is commonly found in the discontinuities of natural rock mass, on the strength of jointed rock is less understood, and thus more high-quality laboratory data to clarify this is necessary (Ramsay, 1980; Cui and Gratchev, 2020).

Well-equipped laboratory tests can provide direct measurements of strength of jointed rock specimens; however, they tend to be rather costly and time-consuming. A numerical model that can accurately estimate the strength of jointed rock may be an alternative solution. Such numerical models can provide valuable insights in the process of crack initiation and development, which will greatly improve our understanding of the failure mechanism in rock. It is noted that few studies (including Tang et al., 2000) involving the finite element method (FEM) have previously been performed using the rock failure process analysis code (RFPA). However, the FEM has some limitations in characterizing rock particle movements during the load application. The discrete element method (DEM),
such as the particle flow method (PFC), has been successfully utilised to estimate the rock strength (Schopfer et al., 2009; Jia et al., 2013; Li et al., 2014; Zhang et al., 2018; Saadat and Taheri, 2019) and to simulate the rock mass failure (Scholtes and Donze, 2013). However, this approach may also have limitations in defining the contact surface of the initial model. The universal distinct element code (UDEC) is another commercially available DEM program commonly used in rock engineering. Unlike other methods, UDEC provides means to consider the discontinuity of rock mass and the joint properties at a large or microscopic level (Itasca, 2011; Wu et al., 2019).

This study seeks to develop a numerical model that can accurately estimate the strength of jointed rock. The UDEC program was used to simulate the behaviour of four different rocks with pre-existing cracks under unconfined compression. Comparisons with the laboratory test results indicated a high level of accuracy of the newly developed model in estimating the UCS of rock specimens with and without infill materials. This numerical model was utilized to explain the behaviour of jointed rock mass during loading and the failure mechanism of the tested rock specimens. This paper presents and discusses the obtained results.

8.2 Methodology

A series of laboratory tests including unconfined compression have been performed to study the effect of pre-existing cracks and infill materials on the strength of sandstone, argillite, and basalt. Two types of sandstone (S1 and S2) whose engineering properties are summarized in Table 8-1 were used. All specimens were of a cylinder shape with a diameter of 50 mm and height of 100mm. The discontinues of either 1cm or 2 cm long at the top and bottom of the specimen (as shown in Figure 8-1b and 8-1c) were produced to study the effect
of cracks on the unconfined compressive strength (UCS) of rock. To better understand the effect of infill, the discontinuities were filled with either sand or clay material. The sand had density of 1.9 g/cm$^3$, and the friction angle of 35°. The clay had the density of 2.2 g/cm$^3$, the friction angle of 27.3°, and cohesion of 17.4 kPa. More details on the experimental procedure can be found in Chen and Gratchev 2020. For each experimental setup, at least 3 tests were performed, and the average values are summarized in Table 8-2.

A numerical analysis was conducted by means of UDEC (Cundall, 1971). The rock block was divided into limited zones (Figure 8-1), which could move and/or rotate individually. For each zone, the behaviour was described as either linear or non-linear based on the selected stress-strain principles.

Figure 8-1 Block and zone distribution (a) and (d): intact sample; (b) and (e): sample with 1 cm crack; (c) and (f): sample with 2 cm crack.
The Mohr-Coulomb model was utilized for each block with the failure equation given in Eq. 8.1:

\[ f_s = \sigma_1 - \sigma_3 \frac{1 + \sin \phi}{1 - \sin \phi} + 2c \sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} \]  

**Eq. 8.1**

Where:

\( \sigma_1 \): maximum principal stress.

\( \sigma_3 \): minimum principal stress.

\( \phi \): friction angle.

\( c \): cohesion.

In addition to the uniaxial strength of the rock block, \( \sigma_c^M \) is defined using the Mohr-Coulomb method as shown in Eq. 8.2:

\[ \sigma_c^M = \sigma_1 - \frac{1 + \sin \phi}{1 - \sin \phi} \sigma_3 \]  

**Eq. 8.2**

The load application was simulated using the strain (displacement) controlled approach (Itasca, 2011; Alshkane et al., 2017). This was performed using the in-build language FISH based on Eqs. 8.3 and 8.4, which are the govern equations for the motion of an individual block (Itasca 2011):

\[ \theta(t + \Delta t) = \theta(t) + \dot{\theta}(t + \frac{\Delta t}{2}) \Delta t \]  

**Eq. 8.3**

\[ x_i(t + \Delta t) = x_i(t) + \dot{x_i}(t + \frac{\Delta t}{2}) \Delta t \]  

**Eq. 8.4**

Where:

\( \theta \): rotation of block about the centroid.

\( x_i \): coordinates of block centroid.

\( u_i \): velocity components of block centroid.

\( t \): time.
The properties of rock used in the numerical analyses were estimated either on the basis of experimental tests and utilizing the reference data from the RockData program of RocScience (Table 8-1). The properties of the infill material such as cohesion, friction angle, and density were obtained from the relevant laboratory tests.

Table 8-1 Properties of rocks

<table>
<thead>
<tr>
<th></th>
<th>Density (g/cm³)</th>
<th>Youngs Modulus (GPa)</th>
<th>Poisson ratio</th>
<th>Cohesion (MPa)</th>
<th>Friction angle (°)</th>
<th>Tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone (S1)</td>
<td>1.78</td>
<td>4.8</td>
<td>0.3</td>
<td>3.5</td>
<td>45.7</td>
<td>8.3</td>
</tr>
<tr>
<td>Sandstone (S2)</td>
<td>2.57</td>
<td>6.6</td>
<td>0.25</td>
<td>10.0</td>
<td>50.0</td>
<td>2.2</td>
</tr>
<tr>
<td>Argillite (A)</td>
<td>2.55</td>
<td>9.4</td>
<td>0.25</td>
<td>7.2</td>
<td>50.4</td>
<td>2.0</td>
</tr>
<tr>
<td>Basalt (B)</td>
<td>2.72</td>
<td>13.9</td>
<td>0.25</td>
<td>2.1</td>
<td>53.6</td>
<td>2.8</td>
</tr>
</tbody>
</table>

8.3 Result and discussion

This section presents and discusses the experimental and numerical results obtained for the specimens of S1 and S2 with pre-existing discontinuities of 1cm and 2cm long. The effect of infill material such as sand and clay on the UCS as well as the common failure patterns of S1 and S2 are also discussed. It is noted that for the argillite (A) and basalt (B), due to limited number of specimens available for laboratory testing, only the effect of 1-cm long crack with the sand as infill was studied. The values of UCS obtained from the laboratory tests and numerical simulations are summarized and compared in Table 8-2.
Table 8-2 Summary of the UCS data obtained from laboratory tests and numerical analysis.

<table>
<thead>
<tr>
<th>Material</th>
<th>Initial conditions</th>
<th>Measured strength from laboratory tests, MPa</th>
<th>Estimated strength from numerical model, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone (S1)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intact</td>
<td></td>
<td>11.2</td>
<td>10.9</td>
</tr>
<tr>
<td>1 cm</td>
<td>None</td>
<td>10.7</td>
<td>10.25</td>
</tr>
<tr>
<td></td>
<td>Sand</td>
<td>13.5</td>
<td>11.1</td>
</tr>
<tr>
<td></td>
<td>Clay</td>
<td>15.1</td>
<td>13.9</td>
</tr>
<tr>
<td>2 cm</td>
<td>None</td>
<td>9.8</td>
<td>9.0</td>
</tr>
<tr>
<td></td>
<td>Sand</td>
<td>13.5</td>
<td>13.0</td>
</tr>
<tr>
<td></td>
<td>Clay</td>
<td>14.9</td>
<td>13.8</td>
</tr>
<tr>
<td>Sandstone (S2)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intact</td>
<td></td>
<td>47.5</td>
<td>46.0</td>
</tr>
<tr>
<td>1 cm</td>
<td>None</td>
<td>41.1</td>
<td>40.0</td>
</tr>
<tr>
<td></td>
<td>Sand</td>
<td>39.5</td>
<td>38.8</td>
</tr>
<tr>
<td></td>
<td>Clay</td>
<td>45.1</td>
<td>43.0</td>
</tr>
<tr>
<td>2 cm</td>
<td>None</td>
<td>30.5</td>
<td>31.6</td>
</tr>
<tr>
<td></td>
<td>Sand</td>
<td>31.9</td>
<td>32.0</td>
</tr>
<tr>
<td></td>
<td>Clay</td>
<td>42.7</td>
<td>43.0</td>
</tr>
<tr>
<td>Argillite (A)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intact</td>
<td></td>
<td>39.4</td>
<td>40.0</td>
</tr>
<tr>
<td>1 cm</td>
<td>None</td>
<td>30.1</td>
<td>30.8</td>
</tr>
<tr>
<td></td>
<td>Sand</td>
<td>31.9</td>
<td>32.2</td>
</tr>
<tr>
<td>Basalt (B)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intact</td>
<td></td>
<td>83.7</td>
<td>88.0</td>
</tr>
<tr>
<td>1 cm</td>
<td>None</td>
<td>69.3</td>
<td>69.0</td>
</tr>
<tr>
<td></td>
<td>Sand</td>
<td>104.3</td>
<td>140.0</td>
</tr>
</tbody>
</table>
8.3.1 Experimental results and numerical analysis for S1

Figure 8-2 presents the results of laboratory tests (a) and the data from numerical analysis (b) in terms of the stress-strain curves. According to the laboratory data, the presence of crack tends to decrease the maximum stress at failure by about 10% for the 1-cm long crack and 20% for the 2-cm long crack. The results of the numerical analysis agree with the experimental data, which shows the same effect of the pre-existing discontinuities on the UCS of S1.

Figure 8-2c, 8-2d and 8-2e shows the vertical displacement estimated by UDEC at the end of the test while Figure 8-2f, 8-2g and 8-2g gives the failure patterns observed in the laboratory tests. A relatively high level of correlation between the numerical and laboratory results can be observed. In particular, for the specimen with an open crack of 1 cm long (Figure 8-2d), the numerical model predicts the initiation of failure from the tip of the pre-existing crack, which was observed at the end of the laboratory test (Figure 8-2g). The same failure pattern was recorded for the specimen with the 2-cm long crack for both numerical (Figure 8-2e) and laboratory (Figure 8-2h) conditions.
Figure 8-2 Stress-strain curves of S1: (a) numerical results, (b) laboratory data.

Vertical displacements estimated by the numerical model and common failure patterns: (c, f) intact specimens, (d, g) specimens with 1 cm long open crack, (e, h) specimens with 2 cm long open crack.
To better understand the failure mechanism, the results of the numerical analysis conducted for the specimen S1 with the 2-cm long crack are given in Figure 8-3. It is noted that the blue colour represents the tension in the specimen, which occurs between the two pre-existing cracks in the early stage of the load application (Figure 8-3a). The appearance of the red colour suggests the compression stresses that begin to form in the specimen (Figure 8-3b), propagating from the tips of the pre-existing cracks (Figure 8-3c). Eventually, this process leads to the shear failure that occurs near the tip of the crack as shown in Figure 8-3d. This failure pattern where a new crack is initiated from the tip of the existing discontinuity has been reported by Yin et al. 2014; Cao et al. 2016; Gratchev et al. 2016; Zhao et al. 2018 for rock-like materials.

Figure 8- 4 The development of X-axis displacements in the intact specimen of S1 (a, b), and the specimen with 2-cm long pre-existing cracks (c, d).

Figure. 4 presents the development of X-axis displacements in the intact specimen of S1 (no pre-existing crack) and in the specimen with 2-cm long open cracks. As shown for
the intact specimen, after the load was applied (Figure 8-4a), the zero horizontal displacement area (green colour) would move from the centre of the block to the side of the block (Figure 8-4b), forming the failure plane. This mechanism was observed during laboratory testing as shown in Figure 8-2c. A different pattern was observed for the specimen with a 2-cm long open crack. Figure 8-4c shows the displacement immediately after the load application, while Figure 8-4d gives the displacement at the end of the simulated test. It is evident from these figures that the zero X-displacement zone (green colour) is still located in between the two pre-existing discontinuities. This implies that the failure plane would likely develop, connecting the tips of the pre-existing cracks. This failure pattern with a tension crack developed in the middle of the specimen was observed in a few laboratory tests including the ones shown in Figure 8-2h (for S1), and Figure 8-7h (for the argillite specimen).

Figures 8-5 demonstrates the effect of infill material on the UCS of S1 obtained from the numerical analysis (a) and laboratory tests (b). It is noted that the values of UCS of S1 with the infilled material estimated using the numerical analysis are very close to the values obtained from the laboratory tests (Table 8-2). As can be seen in Figures 8-5a and 8-5b, regardless of the crack length, the presence of the denser clay infill (compared to the sand infill) leads to the greater values of UCS. It is interesting that the UCS of the specimens with the infilled material was about 10-20% greater compared to the UCS of the intact specimen. This phenomenon was also observed in the laboratory tests. This can be related to the slightly greater strain that the specimens with the infilled material could develop in the unconfined compression, which correlated to greater values of the load that the specimen could withstand before failure. Figures 8-5c, 8-5d, 8-5e, and 8-5f show a good agreement between the numerical analysis and laboratory test results.
Figure 8-5 Strain-stress curves obtained from numerical modelling for S1 with different infill material: (a) 1 cm long crack, (b) 2 cm long crack. Failure patterns from the numerical analysis (c, e) and laboratory tests (d, f).

8.3.2 Experimental and numerical results for S2

Figure 8-6 presents the strain-stress curves of S2 obtained from the numerical model (a), and laboratory tests (b). Both figures show similar behaviour, suggesting a high level of accuracy in the stress-strain prediction from the numerical model. According to the laboratory and numerical results, the presence of open cracks tends to decrease the UCS of S2 by about 15-30%. It is interesting that the infill does not always increase the UCS strength. For S2, the specimens with clay infill do have higher UCS values compared to the open crack specimen. On the contrary, the specimens with the sand infill exhibited lower strength compared to the specimen with no infill. Additionally, the specimens with the sand
infill (1 cm crack and 2 cm crack) showed about 20% and 35% decrease, respectively, compared to the strength of the intact specimen.

Figure 8-6 Strain-stress curves of S2 with (a) numerical simulation, (b) laboratory test.

8.3.3 Experimental and numerical results for argillite

Figure 8-7 compares the results from the numerical analysis (a) and the data from the laboratory tests (b). For both cases, the intact specimen with no pre-existing cracks exhibited the greatest value of UCS. The laboratory data indicates that the pre-existing crack could decrease the UCS by about 20%. Comparisons of the obtained results reveal that the numerical model tends to overpredict the strain at failure. This discrepancy can be attributed to the anisotropic nature of argillite such as foliation, which appears rather difficult to simulate in numerical modelling. However, the failure patterns predicted by the numerical model (Figure 8-7c and 8-7e) seem to be in agreement with the patterns observed in the laboratory tests (Figure 8-7d and 8-7f). In many cases, the failure mechanism involved the new discontinuity that developed from the existing crack to the side of the block, resulting
in shear failure. However, a few cases such as the one shown in Figure 8-7f where additional cracking from the top to the bottom occurred were also observed in the laboratory.

Figure 8- 7 Strain-stress curves obtained for the specimens of argillite: (a) numerical data, (b) laboratory test results. The vertical displacements (c, e) and failure patterns from the laboratory tests (d, f).

8.3.4 Experimental and numerical results for basalt

Figure 8-8 presents the results of numerical simulation (a) and laboratory tests (b) for the basalt specimens plotted as the stress-strain curves. It is evident from this figure that the numerical analysis gives the same relationships which were observed in the laboratory; that is, the existing cracks decreased the UCS of basalt. Also, the sand infill in the existing cracks led to greater UCS values. Both numerical and laboratory results indicate that the specimen with sand infill could undertake larger stains which might result in greater stresses.
at failure. However, it seems that the numerical model overpredicted the UCS of basalt, which can be attributed to the brittle behaviour of this rock. During laboratory testing, the basalt specimens underwent sudden and brittle failure (explosion-like), a behaviour which was rather difficult to incorporate in the numerical model.

![Figure 8-8 Strain-stress curves of Basalt: (a) Numerical results, (b) laboratory results.](image)

### 8.4 Conclusion

A series of laboratory tests and numerical modelling of unconfined compression using UDEC were conducted on four natural rocks (two types of sandstone, argillite, and basalt) with pre-existing discontinuities of either 1 cm or 2 cm long filled with sand or clay. Based on the obtained results, the following conclusions can be drawn:

- The newly developed numerical model was able to estimate the unconfined compressive strength of rock with pre-existing discontinuities to a high level of accuracy. The UCS values and failure patterns estimated by the numerical model agreed with the data obtained in the laboratory.
The experimental results and numerical modelling indicated that the presence of discontinuities (either 1-cm or 2-cm long cracks) would decrease the UCS of natural rocks, while the infill material had the tendency to increase the overall strength of the tested rocks.

The numerical model allowed to identify and explain the mechanism of two common failure patterns observed during laboratory testing. The first pattern was related to shear failure initiated from the existing crack to the side of rock specimen. The second pattern was tension failure that occurred from the tips of the pre-existing discontinuities through the middle of the specimen.

Some discrepancy between the numerical model and lab test results were attributed to the inhomogeneous nature of the natural rocks. In particular, the argillite specimens had foliation anisotropy while the basalt failed in a sudden brittle manner; that is, characteristics of natural rocks which are still rather difficult to incorporate in numerical analysis.

Acknowledgments

This research was performed with the financial support of the Griffith University Postgraduate Research Scholarship (GUPRS).
CHAPTER 9: DISCUSSION AND CONCLUDING REMARKS

9.1 General information

Discontinuities can significantly affect the strength of rock and lead to rock mass instability. To study the strength of jointed rock, a series of unconfined compression and shear tests are typically performed. However, the previous research has mostly related to the strength of rock-like material while the strength properties of jointed natural rocks, including the effect of infill, remains less-explored. For this reason, this study has been performed on natural rocks of different geological origin (three types of sandstone, argillite and basalt), collected from the Gold Coast area, Australia.

The methodology included the laboratory part and numerical analysis. A series of unconfined compression tests have been performed on the natural rock specimens with two types of pre-existing cracks (1 cm long and 2 cm long cracks) and two types of the infill material (sand and clay). In addition, a number of shear tests were performed on two different types of sandstone (S1 and S2) with a different thickness (1mm, 2mm and 3mm) of the infill material. A new numerical model has been developed to accurately predict the strength of jointed rocks and provide insights in the failure mechanism of the rock specimens with pre-existing cracks under the unconfined compression conditions. Based on the obtained results, the major conclusions related to the work’s objectives are given in Chapter 9.2.

9.2 Concluding remarks
9.2.1 Objective 1. To study the effects of discontinuity on the engineering properties of common rocks from SEQ

- To study the effects of discontinuity size and infill on the unconfined compressive strength (UCS) of natural rocks.

The obtained results indicate that the pre-existing discontinuities (i.e., cracks) tend to decrease the UCS of natural rocks. However, the strength reduction seems to depend on the initial strength. It was found that for the specimens with 1-cm long pre-existing cracks, there was about a 13% decrease in the UCS of relatively soft sandstone (S1) (UCS of the intact rock was 10.8 MPa), while greater decreases of 17% and 23% were observed for the relatively harder specimens of basalt and argillite, respectively. It is noted that the initial strength of the intact argillite was 39.9 MPa, and for the basalt, the UCS of intact specimens was about 83.7 MPa.

The infill material had a strong influence on the UCS of natural rocks. It was found that the presence of infill would generally increase the UCS of the tested rocks. In particular, there was an increase of 6% for the argillite specimens, 20% for the basalt specimens, and 35% for the sandstone (S1). These results suggest the greater effect of the infill material on the strength of relatively softer rocks. It was also found that for S3 with 2 cm long cracks, the increase of UCS with sand infill was about 5% and there was an increase of about 40% for the UCS strength with clay infill. It indicated that the denser infill material clay (2.2 g/cm³) can generate a greater influence on the UCS when compared to a loser infill material sand (1.9 g/cm³).

Two major failure patterns observed in the laboratory tests were related to 1) the formation of new discontinuities at the tips of the pre-existing cracks; and 2) the propagation of the newly developed crack/s from the tips of the pre-existing crack to the side of the tested
specimen. The observed failure patterns seem to be similar to the ones reported in the literature for rock-like materials.

- To investigate the influence of discontinuities with infill on the shear strength of jointed natural rocks.

The change in the surface roughness during shearing of jointed natural rocks can be described by the damage coefficient (M). This coefficient was refined in this study to better characterize the changes in surface of natural rocks. It was established that when M<1, no or minimal damage should be expected during shearing, while, when M>1, significant damage to the joint surface may occur. It was also found that M depended on the rock type, and different values of M were proposed for the rocks in this study.

Although the effect of the infill is commonly associated with the decrease of the overall strength of jointed rock, the obtained data indicated that an increase in the infill thickness from 1mm to 3 mm could slightly increase the shear strength of natural rocks. This can be attributed to the rock inhomogeneity which is not possible to observe in the rock-like material.

The critical ratio of infill thickness over the asperity height (t/a) was investigated for two sandstones. It was found that the critical ratios obtained for S1 and S2 were relatively lower (1.2 and 1.5, respectively) compared to the t/a values reported by other researchers for artificial rock-like material such as gypsum plaster and/or cement. It was also established that the t/a ratio may vary due to the irregular shape of the joint surface, which highlights the limitation of the commonly used artificial saw-tooth models.

It was established that the existing methods of shear strength prediction may not accurately estimate the shear strength of jointed rock specimens with irregular surfaces. This study has proposed a new method to consider the interaction between the rock surfaces and the infill material, this providing a more accurate estimation of the shear strength.
9.2.2 Objective 2. To develop a constitutive model which can accurately predict the effect of discontinuities and infill material on the strength of natural rocks

-A new numerical model that can accurately estimate the UCS of different rocks with pre-existing discontinuities were developed and validated against the laboratory data. This model provided valuable insights in the initiation and propagation of failure cracks. The first failure pattern was associated with the shear failure that started from the existing crack and extended to the specimen's side. The second failure pattern was the tension failure, which developed across the middle of the specimen from the tips of the pre-existing discontinuities. Some discrepancies between the numerical model and laboratory test results were attributed to the inhomogeneous nature of the natural rocks. This included the abrupt brittle failure of the basalt and the foliation anisotropy of the argillite, properties of natural rocks which are rather challenging to account for in the numerical analysis.

9.3 Recommendations for future works

In There are recommendations for future research as follows:

- It is recommended to test a range of natural rocks with different geological origin to confirm the findings presented in this study. Also, the influence of different crack geometry and location on the strength of natural rocks should be studied to add more support to the conclusions of this work.
- A numerical study that incorporates the inhomogeneous behaviour of natural rocks is recommended as it would improve the existing methods to predict the strength of jointed rocks. It is also can be suggested that with further field observations and measurements, the model can be more practical with the measurements of different structure and parameters.
As also to the scale-effect, due to the little research of a large-scale model, there are a lot of mysteries which need to be solved. A field observations and measurements can significantly improve the development of any type of numerical model.
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Figure A1- 1 Manual measurements of roughness profiles and its corresponding value of JRC for sandstone samples A1 - A4 before shearing
Figure A1-2 Manual measurements of roughness profiles and its corresponding value of JRC for sandstone samples A5 - A8 before shearing
Figure A1-3 Manual measurements of roughness profiles and its corresponding value of JRC for sandstone samples A9 - A10 before shearing.
Figure A1-4 Manual measurements of roughness profiles and its corresponding value of JRC for sandstone samples A1 - A4 after shearing.
Figure A1-5 Manual measurements of roughness profiles and its corresponding value of JRC for sandstone samples A5 - A8 after shearing
Figure A1-6 Manual measurements of roughness profiles and its corresponding value of JRC for sandstone samples A9 - A10 after shearing
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Figure A2-1 Manual measurements of roughness profiles and its corresponding value of JRC for sandstone 1 sample A1~A10 before shearing.
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Figure A2-4 Manual measurements of roughness profiles and its corresponding value of JRC for sandstone 2 sample B1~B10 before shearing
Figure A2-5 Manual measurements of roughness profiles and its corresponding value of JRC for sandstone 2 sample B1~B10 before shearing