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Shear strength properties of clean and clay infilled rock joints: an analysis of the impast of moisture content under CNL conditions

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SHEAR STRENGTH PROPERTIES OF CLEAN AND CLAY INFILLED ROCK JOINTS: AN ANALYSIS OF THE IMPAST OF MOISTURE CONTENT UNDER CNL CONDITIONS

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ABSTRACT: Rock joints are a type of fracture or discontinuity that have little or no movement that is parallel to the plane of fracture caused by forces acting perpendicular to the fractured walls, where the opening of the break is parallel to the face of least resistance. These are common phenomena in geology around the world and range from small scale to tectonic fault lines. The first part of this project focused on the effect of clean rock joints on the shear strength of the joint at normal loads of 100, 300, 500 and 700 kPa. Rock joints leave an opportunity for infill to occur in the form of soil, water, or mineral precipitates. The second part of this project focuses on the effects of sodium bentonite clay infill with moisture contents of 0%, 10% and 16% and normal stresses of 100, 300, 500 and 700 kPa on the shear strength properties of rock joints. As shear strength is a controlling factor for slope stability, it is important to continue research into this area in order to optimise future engineering project outcomes. There are two main loading conditions for the direct shear testing of rock joints; Constant Normal Loading (CNL) and Constant Normal Stiffness (CNS). As CNL and CNS conditions are representative of different real-world applications it is critical to understand the scope and context of each investigation. This project was conducted under the CNL boundary condition as it predominately focuses on the impact of moisture content on the shear strength of rock joints under unsaturated conditions. These conditions are more likely to occur in shallow rock formations due to the infill and moisture fluctuations that are caused by water infiltration and precipitation.

INTRODUCTION

Improving understanding of the factors that influence engineering decision-making processes is critical to improving safety and engineering project outcomes. Understanding the failure mechanisms and conditions of slope stability is key, both to safety and economic considerations in geotechnical engineering. Rock joints are common geological features with the potential to decrease the shear capacity of the rock material and affect the stability of slopes on small and large scales. Rock joints leave an opportunity for infill to occur. Infill can occur in the form of soil, water, or mineral precipitates. Key variables in rock joint analysis include asperity profile (including profile shape, dimensions, dip angle) infill thickness, infill material, infill moisture content, normal loading, and boundary conditions (CNL or CNS). The aim of this project was to quantitatively analyse the effects of clay infill moisture content and infill layer thickness on the shear strength characteristics of clean and infilled rock joints being subjected to Constant Normal Loading (CNL) boundary conditions. Rock-like samples were constructed using three-dimensional (3D) printed moulds and a cement grout mixture. The scope of this project was limited to:

- one sawtooth (triangular) asperity profile with a base width of 7 mm and asperity height of 2 mm.
- one clay soil type (commercially produced sodium bentonite clay),
- moisture contents of 0%, 10% and 16%,
- applied normal stresses of 100, 300, 500 and 700 kPa subject to CNL boundary conditions, and
- infill thicknesses to asperity ratios of 0 (clean), 0.5, 1.0 and 1.5.

BACKGROUND

Asperity Profile and Joint Modelling

Prior studies have used a range of asperity profiles and specimen sizes including: rectangular specimens (150 mm by 150 mm by 150 mm) with an interlocking sawtooth surface with six teeth across a 150 mm surface with asperity angles of 25°, 40° and 55° (Cao, Deng, Chen & Fu 2018), rectangular

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sawtooth specimens (150 mm by 150 mm by 180 mm total high) with asperity angles of 30˚ and 45˚ and an asperity base length of 20 mm (Jahanian & Sadaghiani 2015), a circular dip-based sawtooth surface rather than a horizontal surface with a 60˚ dip angle and 2 mm asperity height (Indraratna, Jayanathan & Brown 2008), and plain, triangular, and sinusoidal profiles (Zohaib, Mirzaghorbanali, Helwig, Azzia, Gregor, Rastegarmanesh & McDougall 2020).

The joint roughness coefficient (JRC) has been found to be another influencing factor on the design and testing of various asperity profiles. However, as the key objectives of this project were focused on the impact of moisture content, normal loading and infill thickness, a single uniform triangular (sawtooth) asperity profile was be used and remained constant through testing. While JRC has been a focus of some investigations into the shear strength of rock joints, it is not considered a contributing factor in this project as a single joint profile was used throughout the shear testing.

Infill Thickness

The infill thickness to asperity height (t/a) ratio is a common factor of consideration. This relationship is shown in **Figure 1**.

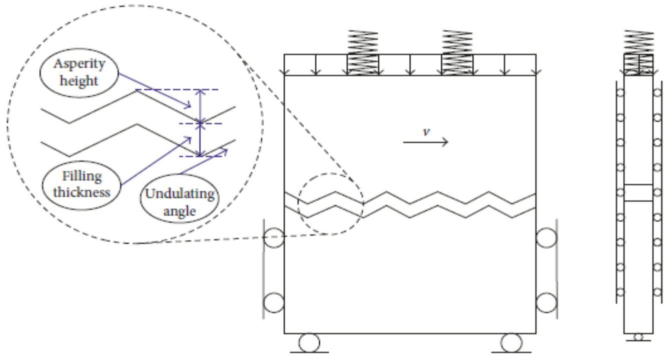

Figure 1: Generalized Testing Model (Wang, Wang & Zhang 2018)

It has been previously determined that the asperity interference influenced the shearing properties at all ratios (Indraratna, Welideniya & Brown 2005). However, at higher infill thicknesses (t/a>1), the impact of the asperity profile on the shear strength of the joint was reduced. The outcomes of Zohaib, Mirzaghorbanali, Helwig, Azzia, Gregor, Rastegarmanesh & McDougall (2020) supported the above conclusion that the thickness of the infill influences the controlling factors that impact shear strength of the joint, and when the infill thickness is greater than the asperity height (i.e. t/a>1), due to no significant friction between the joint surfaces. However, for sawtooth (triangular) joint surfaces where the thickness of infill was less than the asperity height (i.e. t/a<1) both the joint strength and infill influenced the final shear strength. For plain and sinusoidal joint profiles, the presence of infill has minimal impact on the shear strength as there is minimal friction to begin with. This is further supported by Shrivastava & Rao (2017) where they found that an increase in the t/a ratio resulted in a decrease of the shear strength of the rock joint, regardless of the boundary conditions (CNL or CNS) investigated.

Other key outcomes in relation to infill thickness and asperity height included: maximum shear stress occurring at the maximum contact of the joint surfaces (**Figure 2a**) when t/a=0 (clean joint), as the surface contact area decreased the shear stress decreased (**Figure 2b**) until minimum shear stress occurred where the joint was no longer interlocked (i.e. the tips of the joints surfaces were contacting) as shown in **Figure 2c**, increased asperity height resulted in an increased shear stress when maximum contact occurred, where the thickness of infill was equal to the asperity height (i.e. t/a=1) the infill material predominately controlled the shear strength, and as the infill thickness increased the damage to the triangular joint profile decreased due to decreasing contact (Zohaib, Mirzaghorbanali, Helwig, Azzia, Gregor, Rastegarmanesh & McDougall 2020).

Infill Material

Infill material is a common variable to experimental testing, and soils are commonly used for testing to represent real work conditions more accurately. However, natural soil material can be problematic for conceptual investigation as it is highly variable. The introduction of an infill material allows for the surfaces to slide at a lower shear stress due to decreasing the friction between joint surfaces (Zohaib, Mirzaghorbanali, Helwig, Azzia, Gregor, Rastegarmanesh & McDougall 2020); thus, decreasing the shear strength of the rock joint in comparison to clean rock joints of the same surface properties. Infill material used in testing have included: silty-clay (Indraratna, Jayanathan & Brown 2008), sandy-clay (Jahanian & Sadaghiani 2015), bentonite (Indrarathna, Welideniya & Brown 2005), and plaster (Cao, Deng, Chen & Fu 2018).

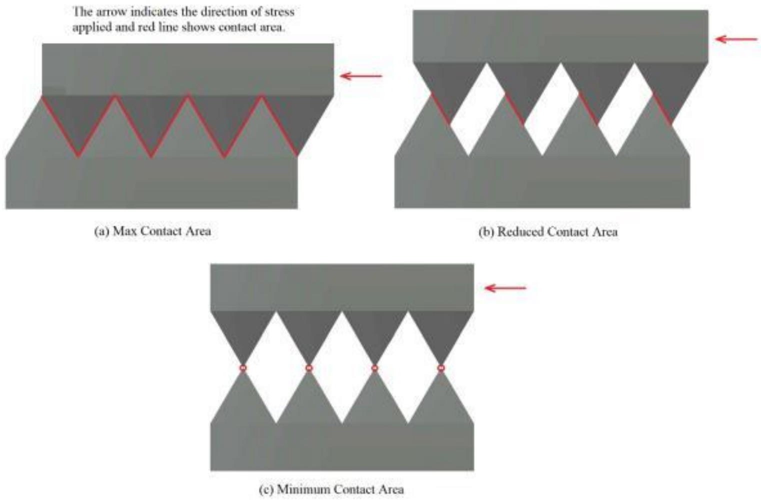

Figure 2: Interlocking Behaviour of Sawtooth Joints (Zohaib, Mirzaghorbanali, Helwig, Azzia, Gregor, Rastegarmanesh & McDougall 2020)

The shear strength of soils is defined as the maximum resistance of soil to shear stress (i.e. the resistance to sliding forces within the soil mass) (Craig 2004, p. 91). If the shear stress exceeds the shear strength of the soil mass then failure occurs. For this project, a sodium bentonite clay infill was used to best represent the behaviour of clay while having the added benefit of being a commercially produced uniform product. The use of a uniform product minimizes the impact of irregularities during the testing procedure and helps to promote a more even saturation effect through the infill material. If a natural clay product were used instead, the results may be impacted by inconsistencies in the soil material and non-moisture related variations between each tests infill.

Moisture Content

As current research on the impact of moisture content on the shear strength of infilled rock joints has been primarily focused on the critical condition of fully saturated soil, this project focused on unsaturated infill conditions. Beyond the optimum moisture content, the cohesion would be expected to decrease due to the increasing water content causing a decrease in the negative pressure of the water filling soil material voids (Craig 2004, p. 4). This, in combination with the findings of the beforementioned study on the effects of infill thickness and asperity profile (Zohaib, Mirzaghorbanali, Helwig, Azzia, Gregor, Rastegarmanesh & McDougall 2020), suggests that as the moisture content increases, the angle of internal friction (φ) will decrease, and once the optimum moisture content is exceeded the shear strength of the infill material will also decrease due to the impact on the cohesion (c) regardless of the infill thickness to asperity height ratio as all investigated ratios are impacted by the infill material. However, it is expected to have a more pronounced effect on ratios of t/a greater than or equal to 1 as the infill characteristic are the primary factors that influence the joints shear strength.

Bentonite clay infill material was used for this project as clay-rich soils are problematic due to their expansive swelling behaviour. Swelling causes changes in the volume of the material in response to the presence of water. Craig (2004, p. 75) defines swelling as a result of the adsorbed water facilitating the clay minerals to undergo “recoverable compression due to increases in interparticle forces” and when the normal stress acting on the soil decreases the solid soil skeleton can expand. This causes a decrease in the pore water pressure, which then increases to a static state as flow within the soil structure occurs, resulting in a decrease in effective normal stress and an increase in volume (Craig 2004, p. 75). In areas of varying precipitation and climate conditions (e.g. extreme wet and dry seasons) this could lead to instabilities in the rock mass and further degradation of existing rock joints in a similar manner to the damaged caused by freeze-thaw processes as investigated by Lei, Lin & Wang (2022).

Normal Loading

Two key testing variables for the direct shear testing of artificial rock joints are CNL and CNS boundary conditions. Liu, Zhu & Li (2020) investigated the differences in artificial rock joints in relation to joint size and shear testing conditions. The conditions used where CNL, CNS, and Constand Normal Displacement (CND). CND is considered a specific case on CNS conditions. When the CNL condition is applied, the normal stress (σn) is held constant and the normal displacement of the shear boxes is servo-controlled (**Figure 3a**) (Liu, Zhu & Li 2020). CNL testing conditions are common in artificial rock joint research. However, previous research in the area has determined that CNL conditions are likely only applicable for shallow rock joints as deeper rock joints are constrained by the surrounding rock material (Liu, Zhu & Li 2020). This is further supported by Han, Jing, Jiang, Liu & Wu (2019) in relation to underground rock joint conditions. Shrivastava & Rao (2015) also support this classification of CNL boundary conditions in real-world applications.

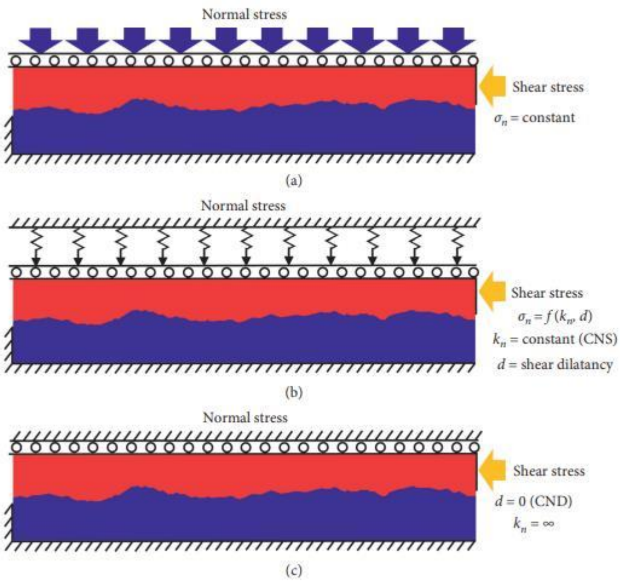

Figure 3: Loading conditions for shear tests a) CNL b) CNS c) CND (Liu, Zhu & Li 2020)

CNS conditions better represent deeper rock joints where the shear dilatancy is constrained (Liu, Zhu & Li 2020). As the normal stress increases, the normal displacement is linearly deformed. Controlling factors of CNS conditions are applied normal stress (σn), normal stiffness of the upper plate (kn), and normal displacement of the upper pressing plate (Δd) where the lower plate is fixed, during period (Δt), as shown in **Figure 3b** and **3c** (Liu, Zhu & Li 2020). CND conditions are a special case within CNS conditions, where the normal displacement (d) is held at zero.

Tang and Wong (2015) investigated the impact of the magnitude of normal loading (0.1 MPa, 0.5 MPa and 1 MPa) on rock joints. This investigation concluded that, as the normal stress increased, the shear strength of the joint increased. Zohaib, Mirzaghorbanali, Helwig, Azzia, Gregor, Rastegarmanesh & McDougall (2020) concluded that the increase in normal loading resulted in an increased maximum shearing stress by inducing higher friction between the two rock joint surfaces.

As CNL and CNS conditions are representative of different real-world applications it is critical to understand the scope and context of each investigation. This project will be conducted under the CNL boundary condition as it predominately focuses on the impact of moisture content on the shear strength of rock joints under unsaturated conditions. These conditions are more likely to occur in shallow rock formations due to infill and moisture fluctuations caused by water infiltration and precipitation. However, if this project were to investigate the impact of moisture content from a water table or aquifer, that would likely result in fully saturated infill material, the CNS boundary condition would be more appropriate due to increased constraint provided by the surrounding rock material at lower depths. A range of normal loading magnitudes were applied as previous research has shown that shear strength can vary under the combination of two variables rather than a single variable in isolation (e.g. the impact of CNS and CNL condition in combination with low and high normal stresses).

Joint Failure

In many studies it was found that the asperity angle and sawtooth height were controlling factors for failure conditions. This is due to the changes in contact area between the two parts of the specimen (Cao, Deng, Chen & Fu 2018). **Figure 4** shows the three phases of shear stress in relation to shear displacement after normal force is applied during testing. Stage 1 involves increasing shear stress while the displacement remains constant. Stage 2 shows a linear relationship between the increasing shear stress and shear displacement commences causing a sliding action between the sawtooth interface. Stage 3 occurs when the tips of the sawtooth interface interact causing a peak in shear stress before failure occurs.

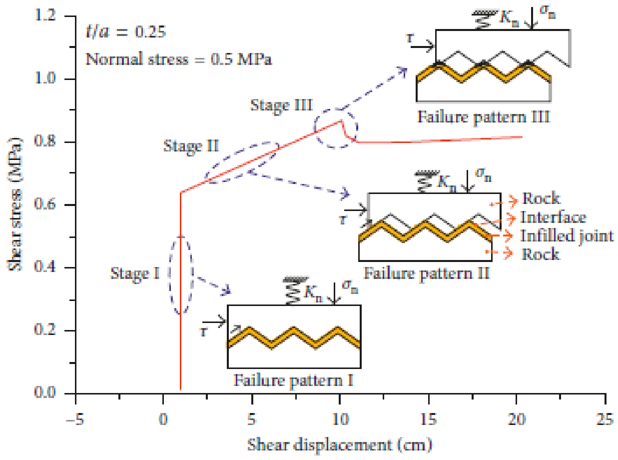

Figure 4: Failure Patterns (Wang, Wang & Zhang 2018)

Failure occurs in three forms: sliding failure, shear failure, and tensile failure (Cao, Deng, Chen & Fu 2018). The asperity angle was found to be a controlling factor in the type of failure mode. Sliding failure occurs when 'the specimen slips along the joint surface and the sawtooth tip wears while it is not sheared' (Cao, Deng, Chen & Fu 2018). Shear failure follows a similar pattern to sliding failure, except the sawtooth tip fails in a horizontal manner due to shear. At asperity angles greater than 55° tensile failure occurs due to increased contact area and resulting frictions, which results in a bending moment at the sawtooth base causing cracking of the asperity and is the most destructive form of failure. All three of these failure modes are destructive in nature and as such further research into rock joint failure is key to predicting and minimizing slope failure.

Dangers of Idealisation

It is crucial to understand the scope, limitations, and applications of this project. As such, the results need to be validated due to the dangers of idealization. Unfortunately, laboratory testing results do not always reflect the mechanisms in real world scenarios. Barton (2013) outlines this issue and warns that 'simplicity is hardly a substitute for reality'. While idealized testing parameters are key to controlling input variables in order to better understand the mechanical relationships, it is important that these relationships are understood in detail before they are implemented in field practices.

Numerous studies on the topic of shear strength of clean and infilled rock joints have been completed. However, these studies largely focus on the variable of infill thickness or type under critical conditions (full saturation). While it is important to understand how an infilled rock joint behaves under critical conditions, it does not promote a quantitative understanding of how the moisture content impacts the shear strength parameters of infill with varying moisture contents. As such this project will compare the effects of infill moisture content, infill thickness and normal loading on the shear strength of the rock joint and contextualise these outcomes with those of past research.

EXPERIMENTAL DESIGN

Mould Design, Formation and Preparation

The final specimen required a diameter of 63 mm and overall height of 36 mm based on ShearTrac2 testing equipment requirements. This diameter was used to determine the internal diameter of the casting mould, and this overall height must include the top and bottom interlocking cement grout rock joints and the thickness of the infill material. As the key focus of this project is the impact of moisture content on the shear strength, an asperity angle of less than 30° is used to minimise the impact of joint friction and aim for Type 1 sliding failure rather than shear or tensile failure of the rock joint. As such, a single uniform triangular (sawtooth) asperity profile will be used with an asperity height of 2 mm, asperity length of 7 mm, and asperity angle of 29.7°. Autodesk Fusion360 was used to model the interlocking asperity profile. The final 3D modelled profile is shown in **Figure 5**. The profile was printed on the *Anycubic Mono X (4K) SLA* printer using Anycubic's standard grey UV resin at a layer height of 50 microns.

Figure 5: 3D model of interlocking sawtooth mould

Polyvinyl chloride (PVC) Drain waste & Vent (DWV) pipe (nominal diameter of 65 mm (DN65)) was cut at 19 mm lengths to form the vertical component of the mould. The asperity mould was fixed to the end of the 19 mm lengths of PVC pipe to form the moulds for the cement grout 'rock joint' (**Figure 6**). As this PVC pipe has an internal diameter of 62 mm, the pipe was split vertically to facilitate the 63 mm diameter of the asperity mould. This, in combination with applying all-purpose industrial grease to the internal surfaces of the mould, allowed for easy removal of the cast specimen after an initial setting time. The grease on the internal surfaces of the mould was applied with care to avoid impacting the desired profile of the mould. In addition, five 3-part cubic moulds (each part was 40 mm by 40 mm by 40 mm) were oiled, and used to cast samples for uniaxial compression testing to determine the compressive strength of the cement grout mix.

Figure 6: Greased artificial rock joint moulds

Casting

A custom mix of 0.2 to 0.6 mm sand (900 g), Portland cement (300 g) and potable water (180 ml) was used. The cement grout mixture was made in batches to improve workability. Immediately after each batch was poured each mould was manually agitated to release trapped air bubbles and liquify the mixture to allow for easier flow into the asperity profile. Care was taken to not damage the 3D printed mould profile during this process. This process was repeated until all moulds were filled. The artificial rock joints were left for 36 to 48 hours to set before being demoulded and labelled '1' (top portion of the joint (**Figure 7**)) and '2' (bottom portion of the joint (**Figure 7**)). The rock joint samples were then left to cure for 28 days in a temperature and humidity-controlled curing room.

Figure 7: Demoulded Sample top (left) bottom (right)

Compression Testing

Using the 40 mm by 40 mm by 40 mm cubes the uniaxial compressive strength of the cement grout mix was investigated. The uniaxial compression testing was run at a loading rate of 0.5 kN/s and the maximum compressive strength recorded. This testing was completed at 7, 14, 21 and 28 days, with a minimum of three samples on each occasion to determine to average compressive strength of the cement grout mix. The uniaxial compression testing at 7, 14, 21 and 28 days resulted in average compressive strengths of 15.583, 12.604, 15.922 and 16.325 MPa respectively. As the uniaxial compressive strength of the artificial rock specimen is less than 20 MPa, it is classed as a 'soft rock' (Agustawijaya 2007).

Infill Material Testing

Preliminary soil material testing was undertaken on the commercially produced Sodium Bentonite clay material. A summary of the soil property values shown in **Table 1**.

Table 1: Infill Material Properties Results Summary

Infill Property	Result	Relevant Standard
Particle Size Distribution (d)	45 microns	AS1289.3.6.1-2009
Infill Particle Density (ρ_s)	2125 kg/m^3	AS1289.3.5.1-2006
Moisture Content (w_L)	16.1%	AS1289.2.1.1-2005
Liquid Limit (LL)	168	AS1289.3.9.1-2015
Plastic Limit (PL)	89.5	AS1289.3.2.1-2009
Shrinkage Limit (SL)	35.0	From Plasticity Chart
Compression Index (C_C)	0.609 (w=127.4%)	AS1289.6.6.1-2020
Swell Index (C_S)	0.113 (w=127.4%)	AS1289.6.6.1-2020
Friction Angle (φ')	86˚	AS1289.6.2.2-2020
Cohesion (c')	14.632 kN/m^2	AS1289.6.2.2-2020

Direct Shear Testing of Rock Joints

Direct shear testing of the rock joints was undertaken, applying CNL conditions, using a ShearTrac2 shear apparatus. The key independent variables of interest of this project were the infill thickness to asperity height ratio (t/a), moisture content, and normal loading. Four infill thickness to asperity height ratios of 0 (clean joints), 0.5, 1, and 1.5 were used. Four normal loading conditions of 100, 300, 500 and 700 kPa were used. Three moisture content conditions of 0%, 10% and 16% were used. This totalled 40 rock joint direct shear tests. Each test was coded according to the testing variables where the first number represented the loading case, the second number represented the t/a ratio case, the third number represented the moisture content case, and CL represented the clean joints. This facilitated simple tracking of the test conditions and data collation. The ShearTrac2 Shear Apparatus (**Figure 8**) uses 'two force transducers (horizontal and vertical) and two displacement transducers (horizontal and vertical) to offer feedback to provide real-time control by computer' (Geocomp 2021). The ShearTrac2 also features servo control motor systems facilitating high precision data outcomes.

The bottom half of each artificial rock joint profile was sanded until the top of the asperity sat flush with the top of the bottom half of the split box (with the base plate installed (no porous stone)) (**Figure 9**). The top half of each artificial rock joint profile was sanded to the appropriate height (taking into account the infill thickness) leaving a 7 mm gap from the top of the joint to the top of the shear box. In addition, a line was added along the flat surface of the top joint indicating the direction of the joint profile to allow for easier alignment when constructing the sample for testing. Direct shear testing of the rock joints was completed for each of the conditions outlined in **Table 2**. Testing was completed at a shear rate of 0.5 mm/minute with read times at 0.25 minute steps. Due to the limitations of the machine used, thresholds of 2200 N (shear or normal loads) and 15 mm horizontal moment were input as the maximum values for testing, and the test was aborted if either of these threshold was reached.

Figure 8: ShearTrac2 Shear Apparatus

Figure 9: Alignment of bottom joint in shear box

Table 2: Testing Conditions

Test Code	Loading (kPa)	t/a Ratio	Infill Thickness (mm)	Moisture Content (%)
CL1	100	0	0	Not Applicable (Clean Joint)
CL2	300	0	0	
CL3	500	0	0	
CL4	700	0	0	
111	100	0.5	1	Condition 1 (16%)
211	300	0.5	1	Condition 1 (16%)
311	500	0.5	1	Condition 1 (16%)
411	700	0.5	1	Condition 1 (16%)
112	100	0.5	1	Condition 2 (0%)
212	300	0.5	1	Condition 2 (0%)
312	500	0.5	1	Condition 2 (0%)
412	700	0.5	1	Condition 2 (0%)
113	100	0.5	1	Condition 3 (10%)
213	300	0.5	1	Condition 3 (10%)
313	500	0.5	1	Condition 3 (10%)
413	700	0.5	1	Condition 3 (10%)
121	100	1.0	2	Condition 1 (16%)
221	300	1.0	2	Condition 1 (16%)
321	500	1.0	2	Condition 1 (16%)
421	700	1.0	2	Condition 1 (16%)
122	100	1.0	2	Condition 2 (0%)
222	300	1.0	2	Condition 2 (0%)
322	500	1.0	2	Condition 2 (0%)
422	700	1.0	2	Condition 2 (0%)
123	100	1.0	2	Condition 3 (10%)
223	300	1.0	2	Condition 3 (10%)
323	500	1.0	2	Condition 3 (10%)
423	700	1.0	2	Condition 3 (10%)
131	100	1.5	3	Condition 1 (16%)
231	300	1.5	3	Condition 1 (16%)
331	500	1.5	3	Condition 1 (16%)
431	700	1.5	3	Condition 1 (16%)
132	100	1.5	3	Condition 2 (0%)
232	300	1.5	3	Condition 2 (0%)
332	500	1.5	3	Condition 2 (0%)
432	700	1.5	3	Condition 2 (0%)
133	100	1.5	3	Condition 3 (10%)
233	300	1.5	3	Condition 3 (10%)
333	500	1.5	3	Condition 3 (10%)
433	700	1.5	3	Condition 3 (10%)

RESULTS AND DISCUSSION

Shear Testing of Rock Joints

The peak-trough behaviour with a wavelength approximately equal to the asperity base length of 7 mm (**Figures 10** and **11**) supports the expected outcomes as based on previous research for clean and infilled joints as the infill thickness is less than the asperity height. In addition, this data also reflected the expected outcome that as the infill thickness increases to equal or greater than the asperity height, the infill properties would become the dominant controlling factor. In **Figures 11** and **12**, as the moisture content increased the convergence rate of the shear stress increased as shown by the decreasing amplitude. In addition, the magnitude of the amplitude decreased as the shear displacement increased (due to the decrease in surface contact area as each sawtooth passed beyond the confine of the initial 63 mm cylindrical sample). This was not directly reflected in the trends of the data for higher applied normal stress (500 and 700 kPa) due to limiting conditions as shown in **Figures 13** and **14** (due to increasing normal stress resulting in increasing shear stress, which quickly reached the upper testing limit). However, there were indicators throughout the analysis that could support this trend regardless of the applied normal stress. As such, a key conclusion can be drawn that, as the moisture content increases, the rate of convergence to the trending shear stress increases and the trending shear stress is generally reflective of the maximum shear stress of the infill material (thus the convergence is more pronounced when the infill thickness is greater than the asperity height). However, the magnitude of this convergent shear stress appears to be largely unaffected by the moisture content or infill thickness conditions, but rather controlled by the applied normal stress. Further testing at normal stresses lower than 300 kPa with a wider variety of moisture contents could prove to validate these results.

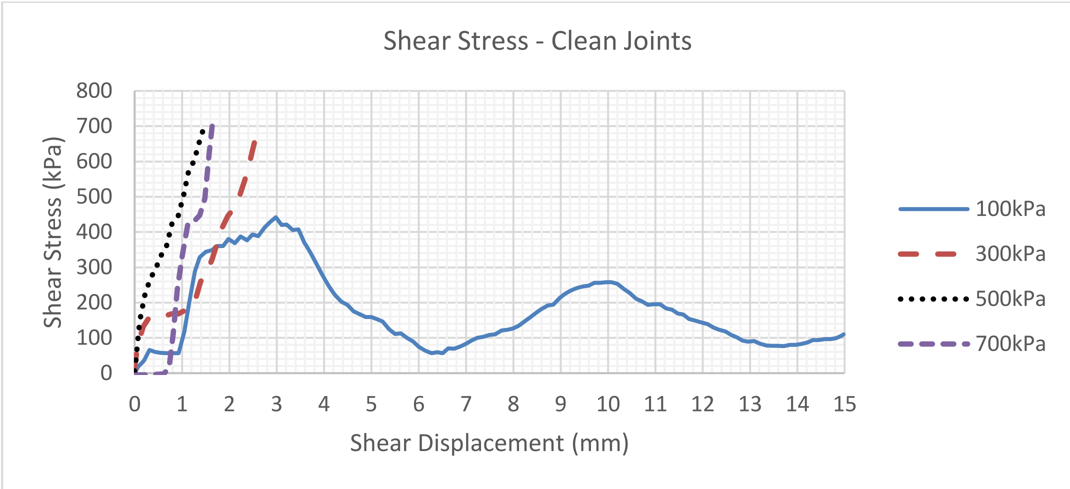

Figure 10: Clean Joint Results (t/a=0) (Shear Displacement vs Shear Stress)

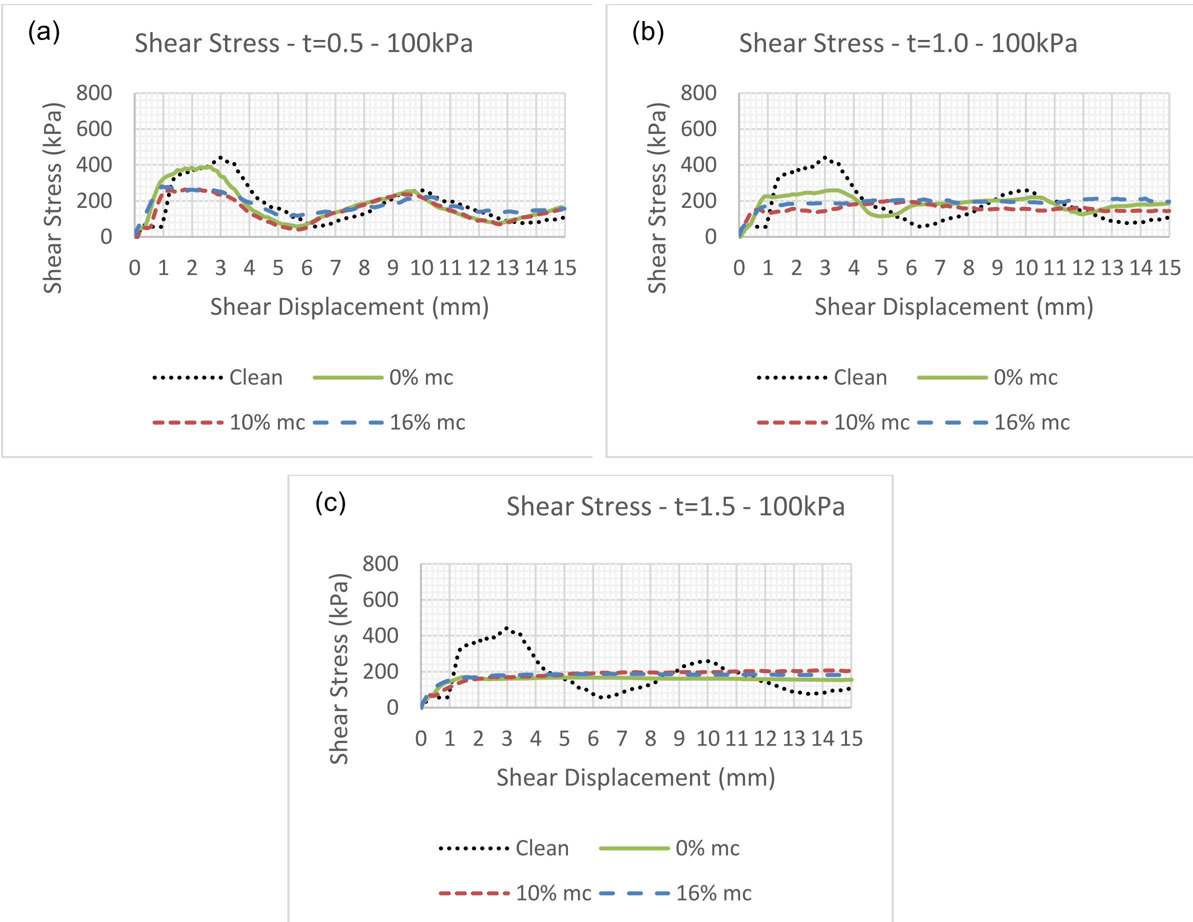

Figure 11: Shear Testing Results for 100 kPa (Shear Displacement vs Shear Stress)

a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

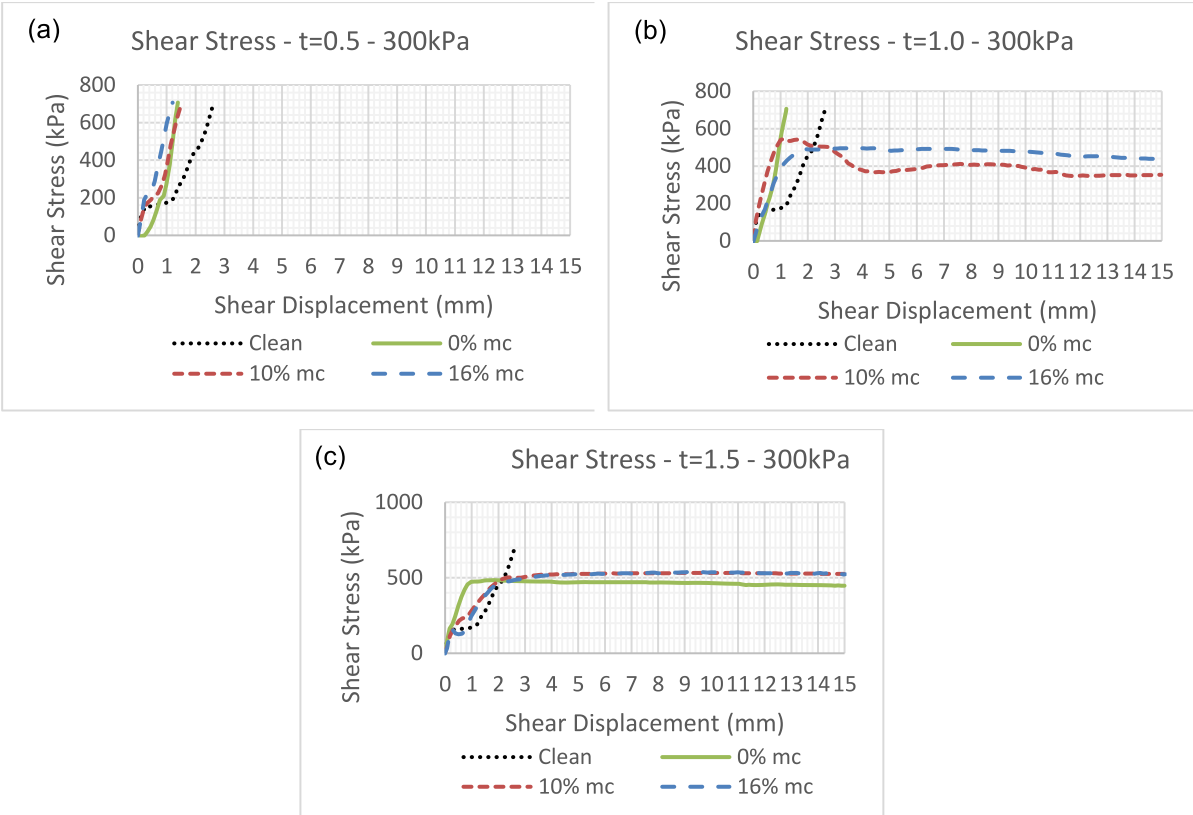

Figure 12: Shear Testing Results for 300 kPa (Shear Displacement vs Shear Stress) a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

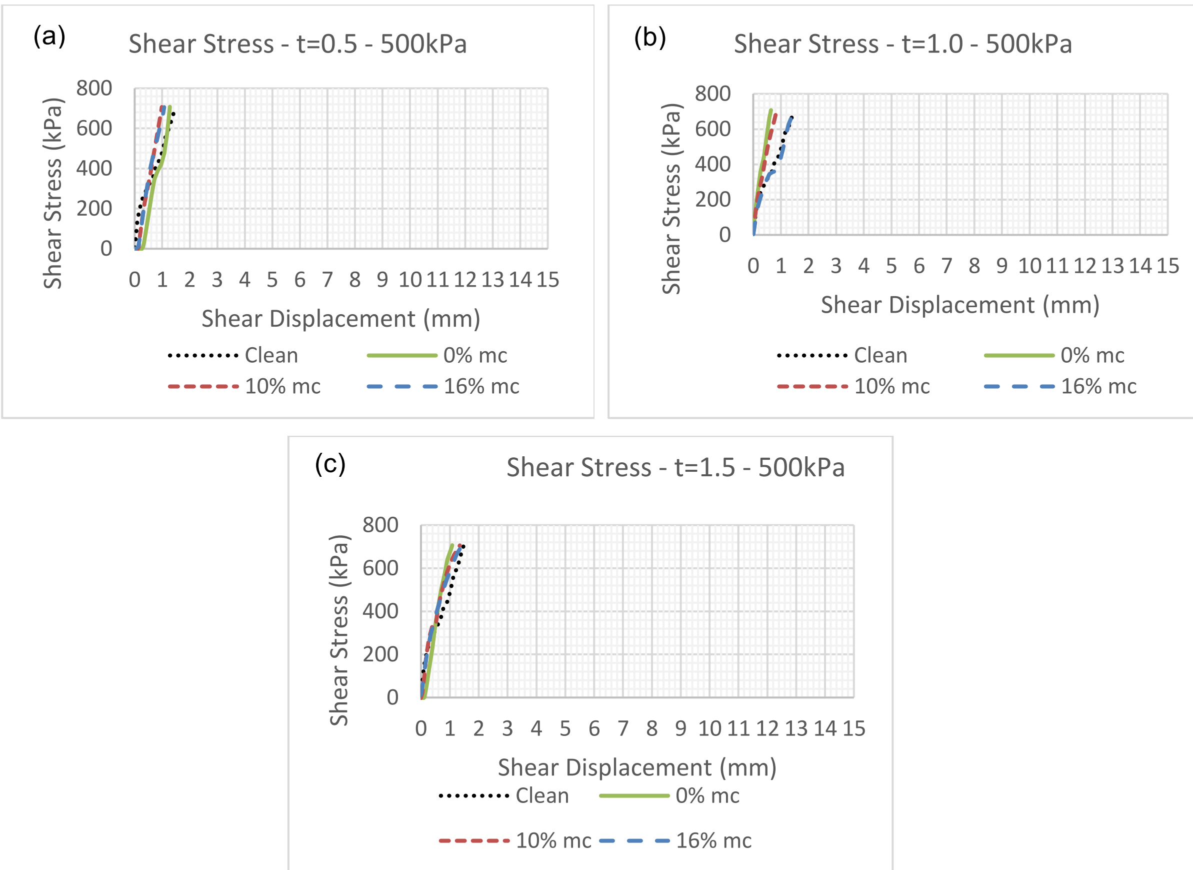

Figure 13: Shear Testing Results for 500 kPa (Shear Displacement vs Shear Stress) a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

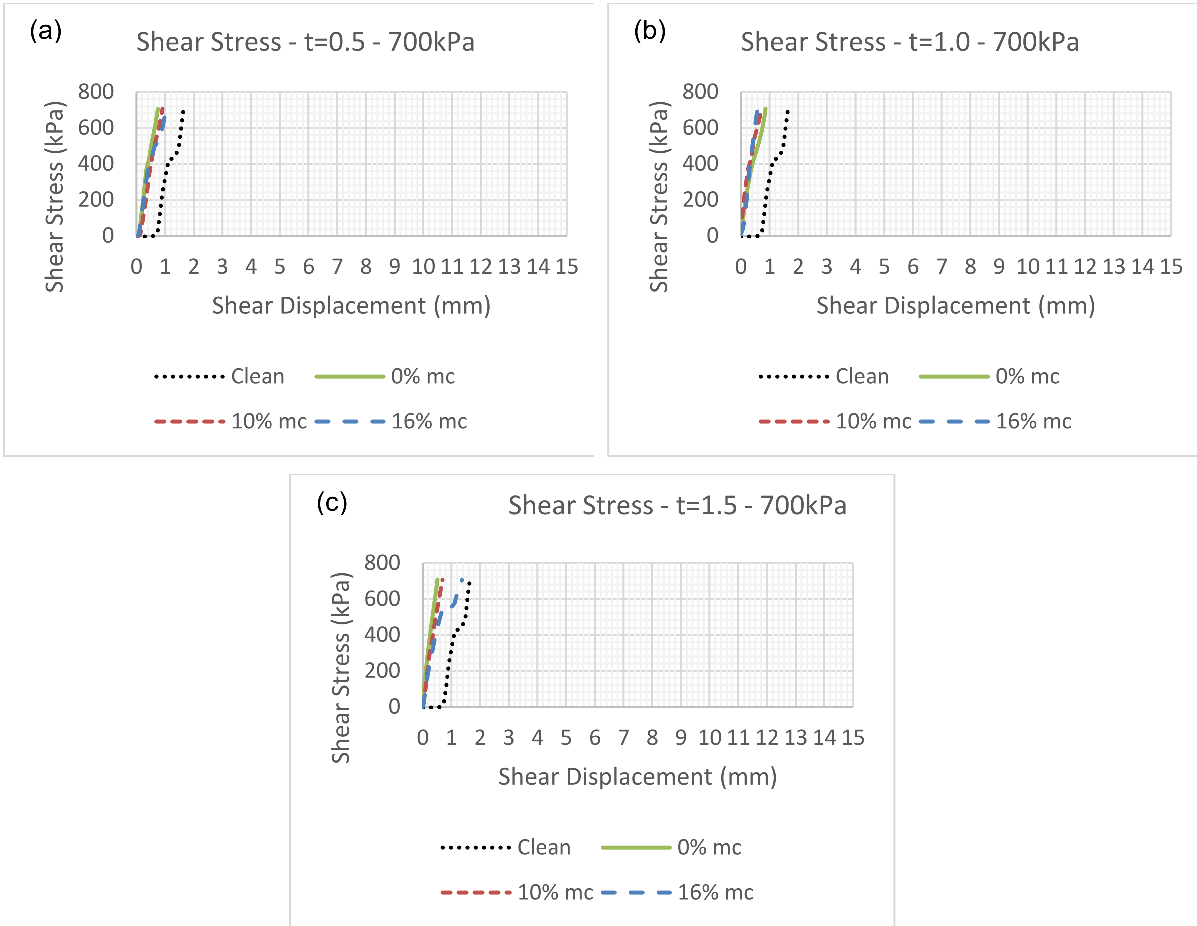

Figure 14: Shear Testing Results for 700 kPa (Shear Displacement vs Shear Stress) a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

Shear Displacement versus Normal Stress

The peak-trough behaviour with a wavelength approximately equal to the asperity base length of 7 mm (**Figures 15** and **16**) supports the expected outcomes based on previous research for clean joints and joints where the infill thickness is less than the asperity height. In addition, this data also reflected the expected outcome that as the infill thickness increases to equal or greater than the asperity height, the infill properties would become the dominate controlling factor. In **Figure 16**, as the moisture content increased the convergence rate of the normal stress increased as shown by the decreasing amplitude of the wave form. Where the magnitude of the amplitude decreased with increasing shear displacement in **Figures 10**, **11** and **12** with regards to shear stress, this does not occur in the normal stress data in **Figures 15**, **16** and **17** as the normal stress applied is constant due to CNL boundary conditions and servo controlled by the ShearTrac2. In **Figures 18** and **19**, this is also reflected in the trends of the data for higher applied normal stress. Based on the data available in **Figure 16** (to a lesser extent **Figure 17**), the conclusion can be drawn that, as the moisture content increases, the rate of convergence to the applied normal stress increases, which is more pronounced when the infill thickness is greater than the asperity height. However, due to the small stress fluctuations of up to approximately ±5 kPa further testing with a wider range of moisture contents at a low applied normal stress (e.g. 100 kPa), with targeted analysis of the normal stress trends could show the trends more conclusively. In addition, testing under CNS conditions with varying moisture contents would provide better insight, into the impact of moisture content on the normal stress of rock joint subjected to shear, due to the release of normal stress as a fixed boundary condition.

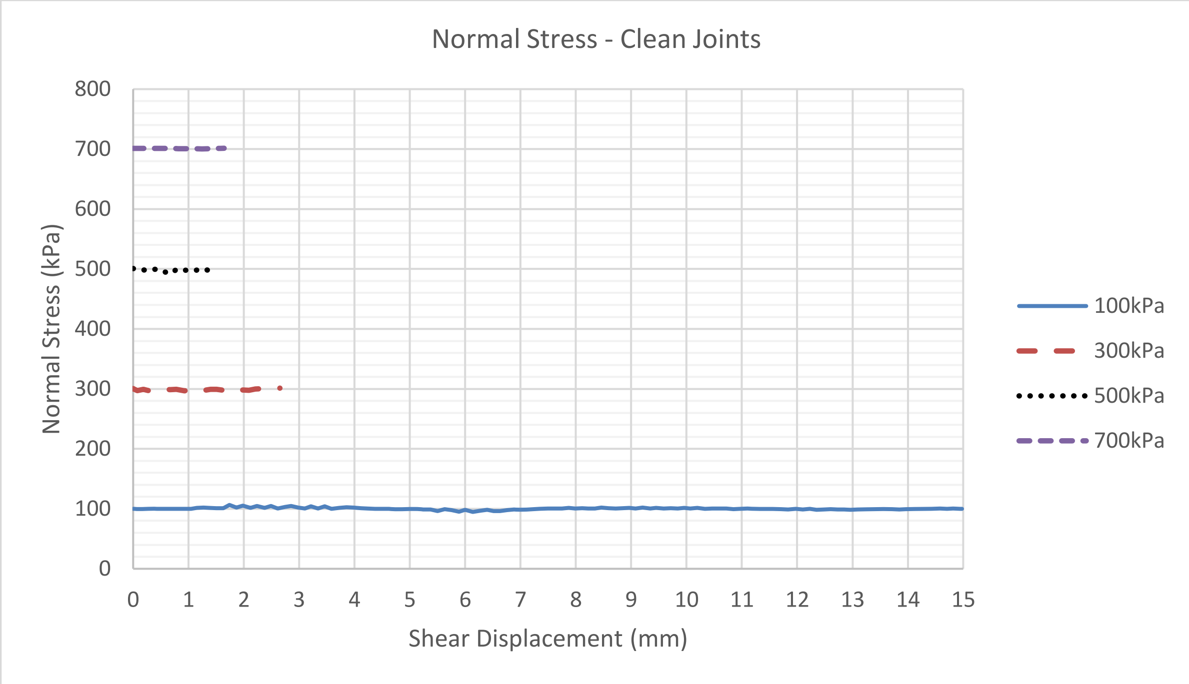

Figure 15: Clean Joint Results (t/a=0) (Shear Displacement vs Normal Stress)

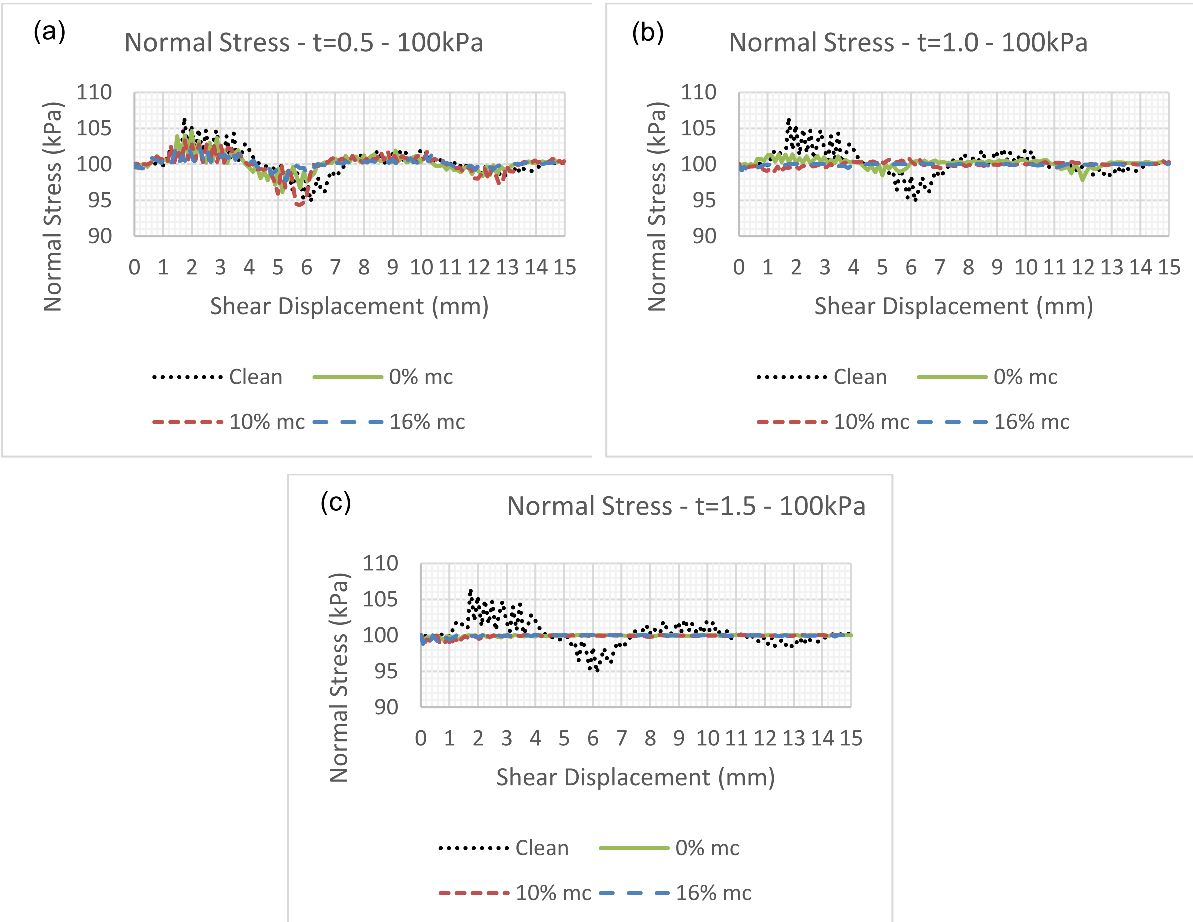

Figure 16: Shear Testing Results for 100 kPa (Shear Displacement vs Normal Stress)
a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

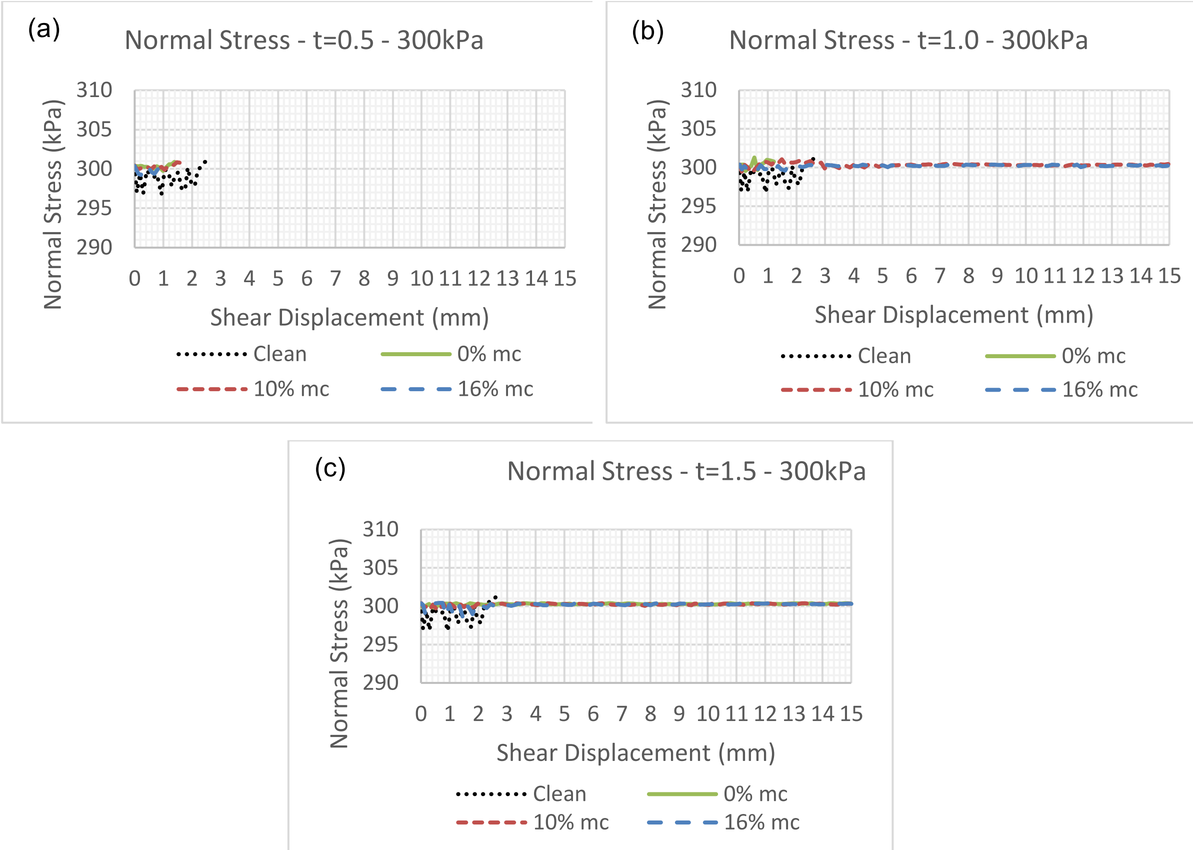

Figure 17: Shear Testing Results for 300 kPa (Shear Displacement vs Normal Stress) a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

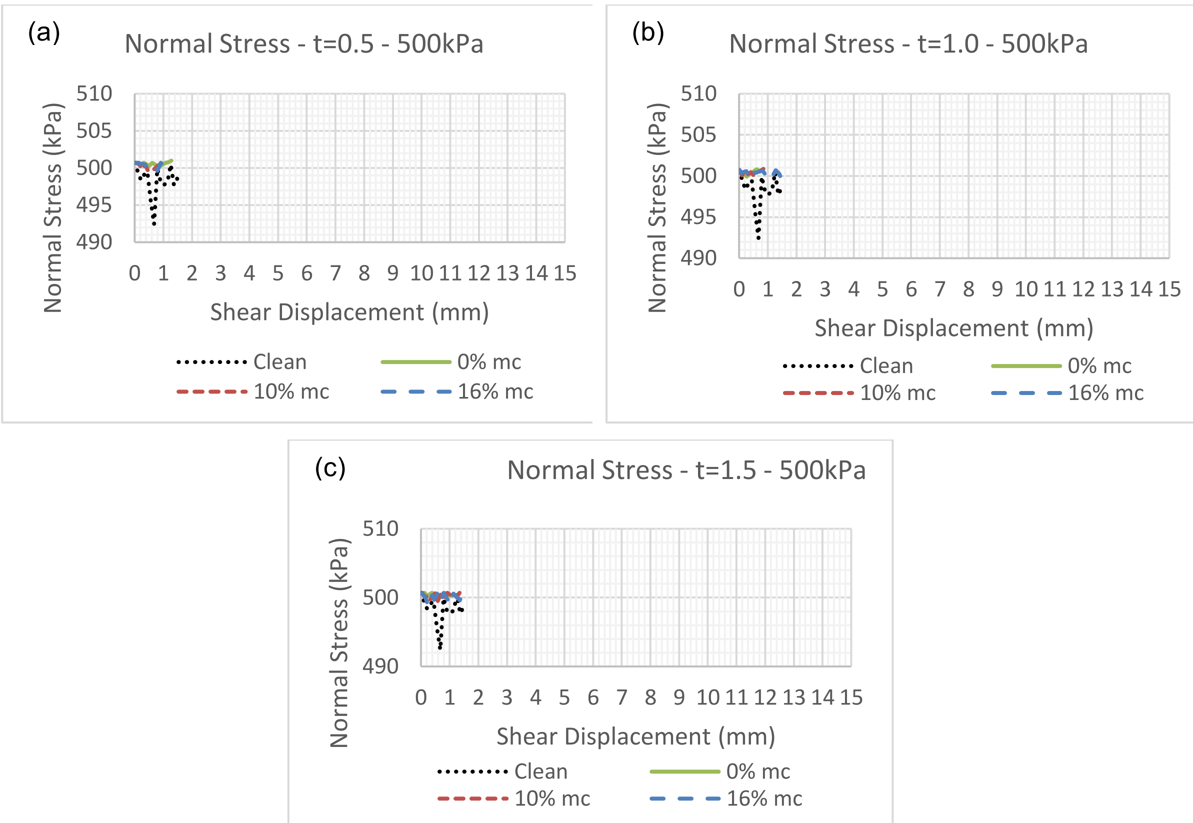

Figure 18: Shear Testing Results for 500 kPa (Shear Displacement vs Normal Stress) a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

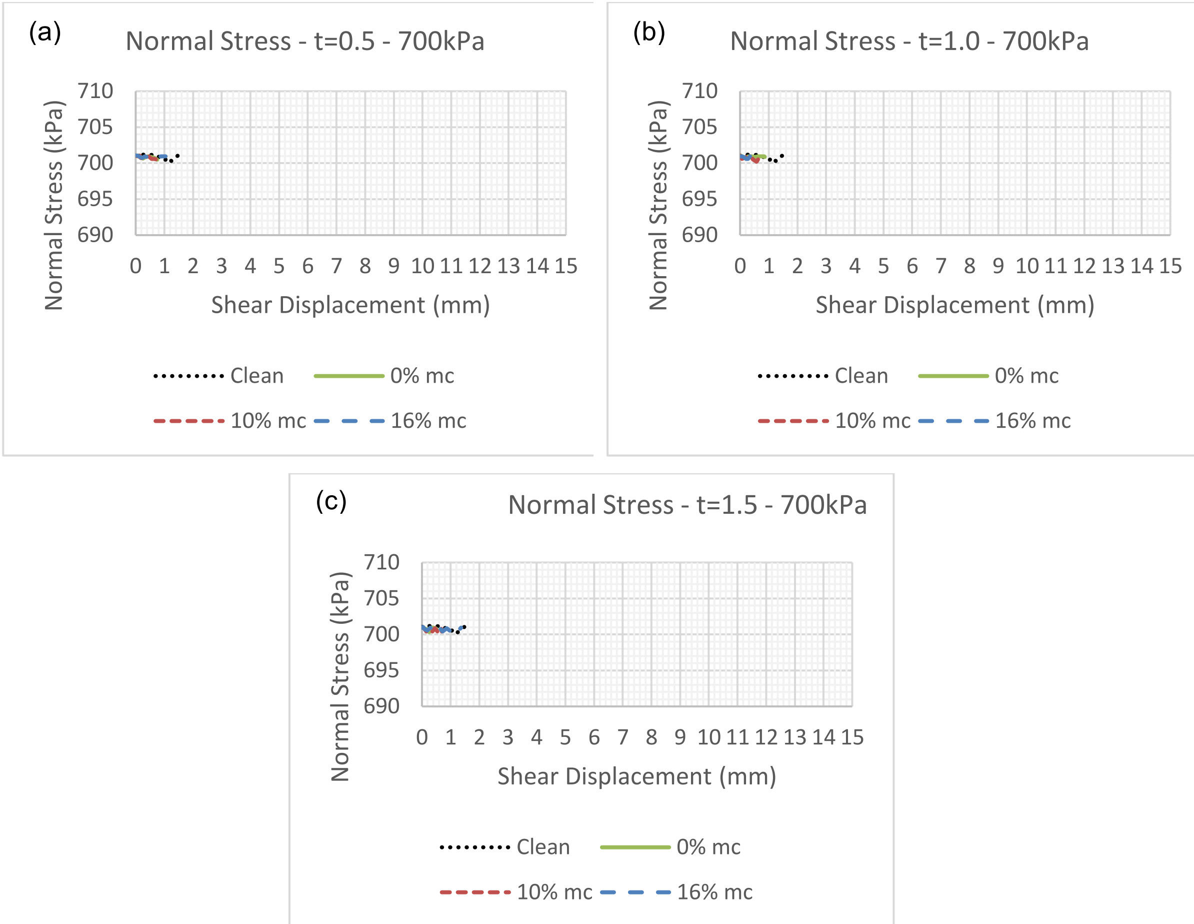

Figure 19: Shear Testing Results for 700 kPa (Shear Displacement vs Normal Stress)a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

Shear Displacement versus Normal Displacement

The peak-trough behaviour with a wavelength approximately equal to the asperity base length of 7 mm (**Figures 20** and **21**) supports the expected outcomes based on previous research for clean joints and joints where the infill thickness is less than the asperity height. In addition, this data also reflected the expected outcome that as the infill thickness increases to equal or greater than the asperity height, the infill properties would become the dominate controlling factor. In **Figure 21**, as the moisture content increased, the convergence rate of the normal displacement increased as shown by the decreasing amplitude. Also, the amplitude decreased with increasing shear displacement in **Figure 21**. This indicates that as each asperity step sliding failure (as shown in **Figure 22**) occurred the asperity height was reduced, and the valleys of the asperity profile were infilled with failed rock joint material. In **Figure 23**, where the infill thickness is equal to the asperity height peak-trough pattern reoccurs for the moisture content of 10% and 16%, before starting to converge on a common contraction value. Based on the data available in **Figure 21** (to a lesser extent **Figure 23**), the conclusion can be drawn that as the moisture content increases, the rate of convergence to the normal displacement trend increases, which is more pronounced when the infill thickness is greater than the asperity height. An additional conclusion is that as the infill thickness increases (causing infill dominated control of shear strength parameters), contraction rather than dilation occurs. Dilation trends only occurred when the surface friction between the joint surfaces, and the soil particles, is highest (clean joints and at 0% moisture content combined with low infill thickness). This is likely due to the lubricating effect of moisture on the infill material. With decreased moisture content the angle of internal friction of the infill increased, thus the friction forces acting on the infill soil particles increased. However, when the moisture content increases the particles can more easily slide past each other, thus facilitating increased contraction.

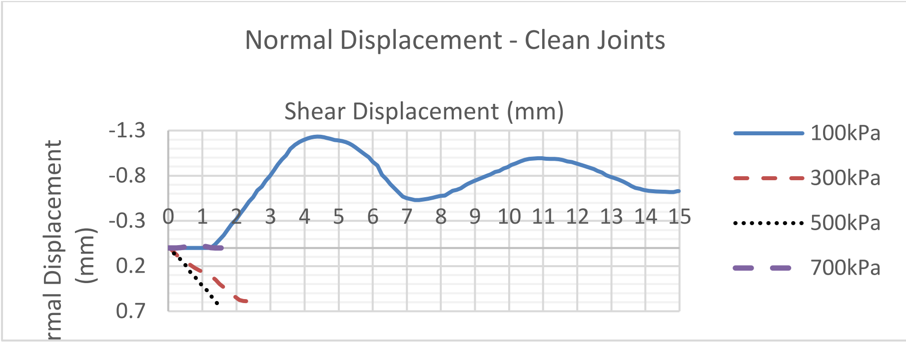

Figure 20: Clean Joint Results (t/a=0) (Shear Displacement vs Normal Displacement)

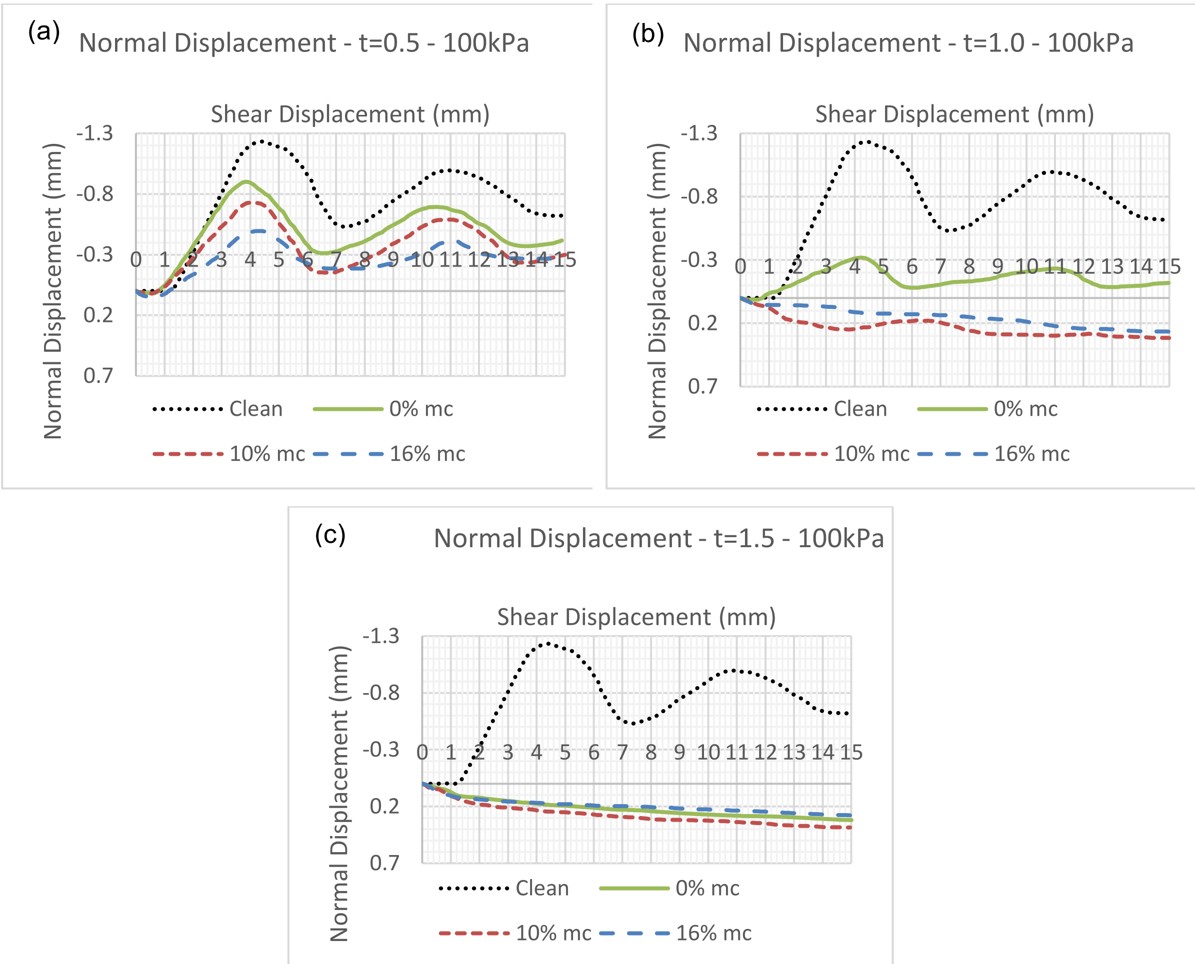

Figure 21: Shear Testing Results for 100 kPa (Shear Displacement vs Normal Displacement) a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

Figure 22: Joint Failure on Sample CL1

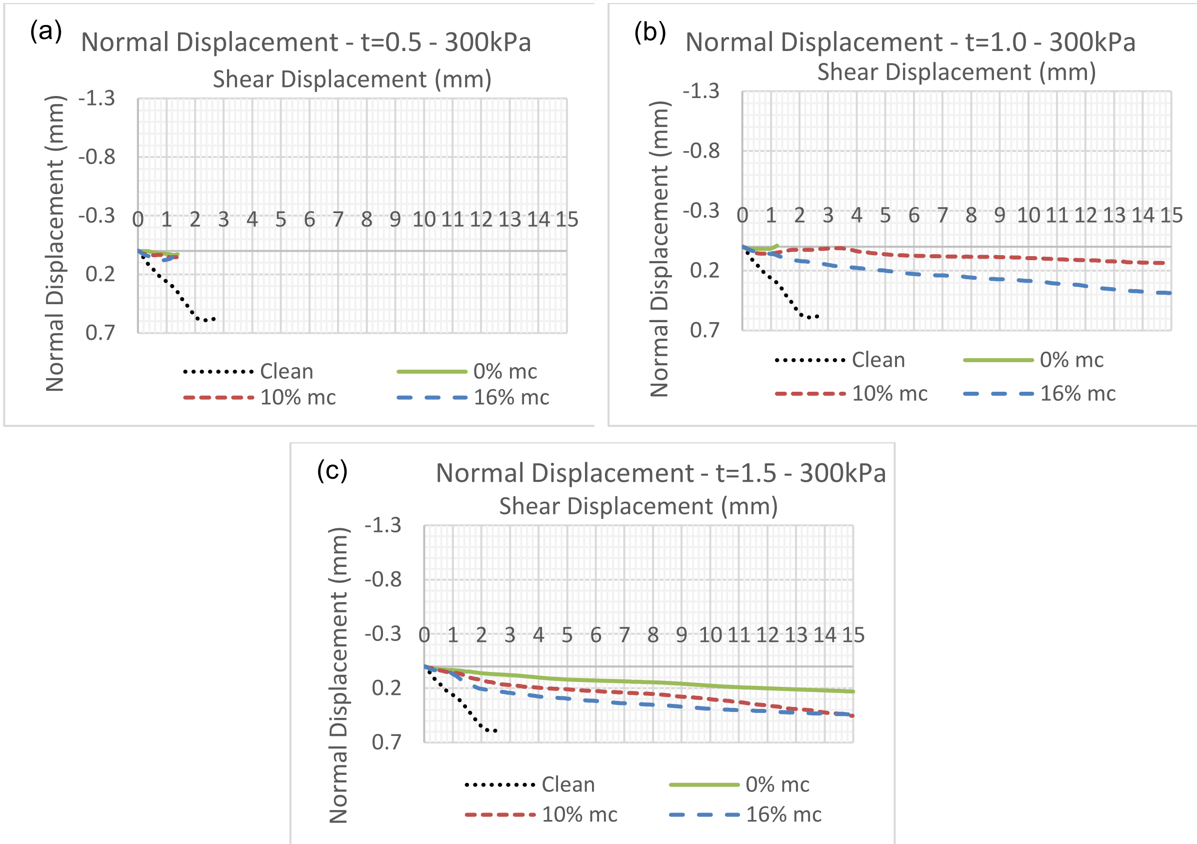

Figure 23: Shear Testing Results for 300 kPa (Shear Displacement vs Normal Displacement) a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

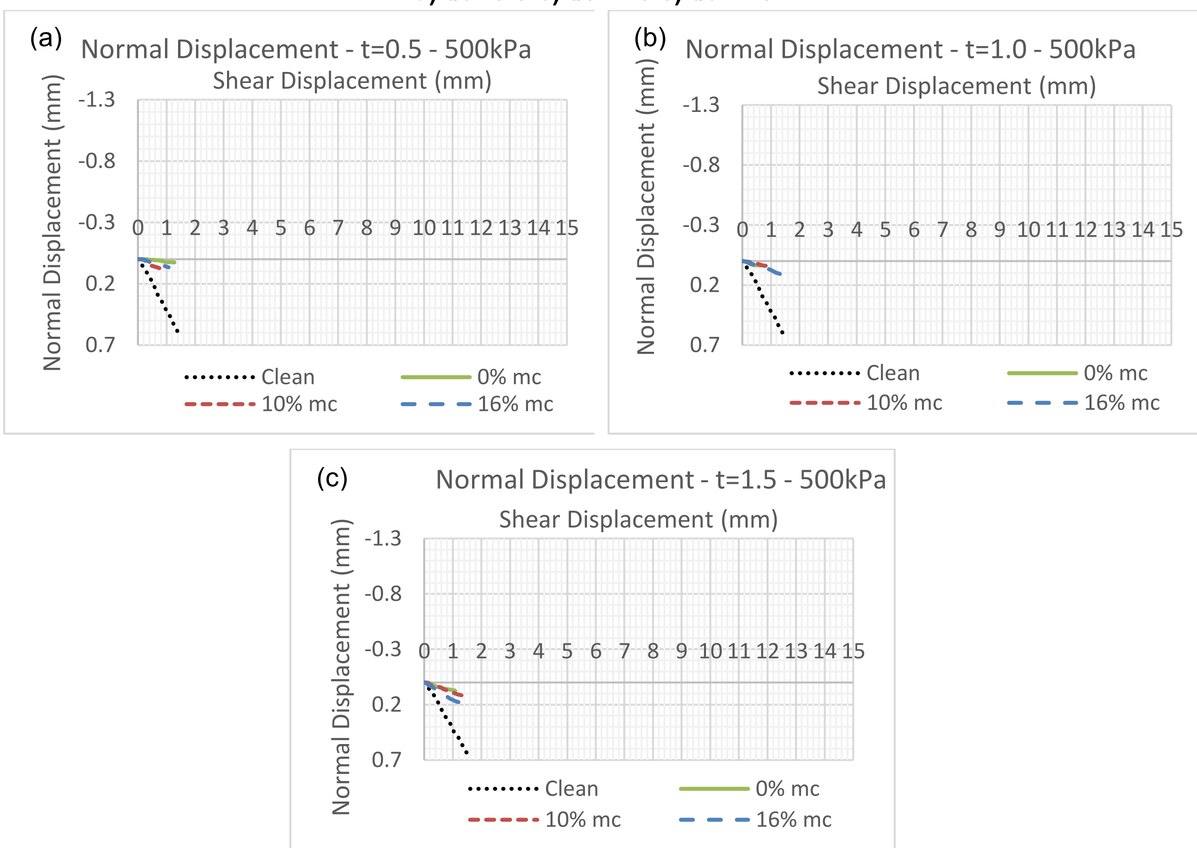

Figure 24: Shear Testing Results for 500 kPa (Shear Displacement vs Normal Displacement) a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

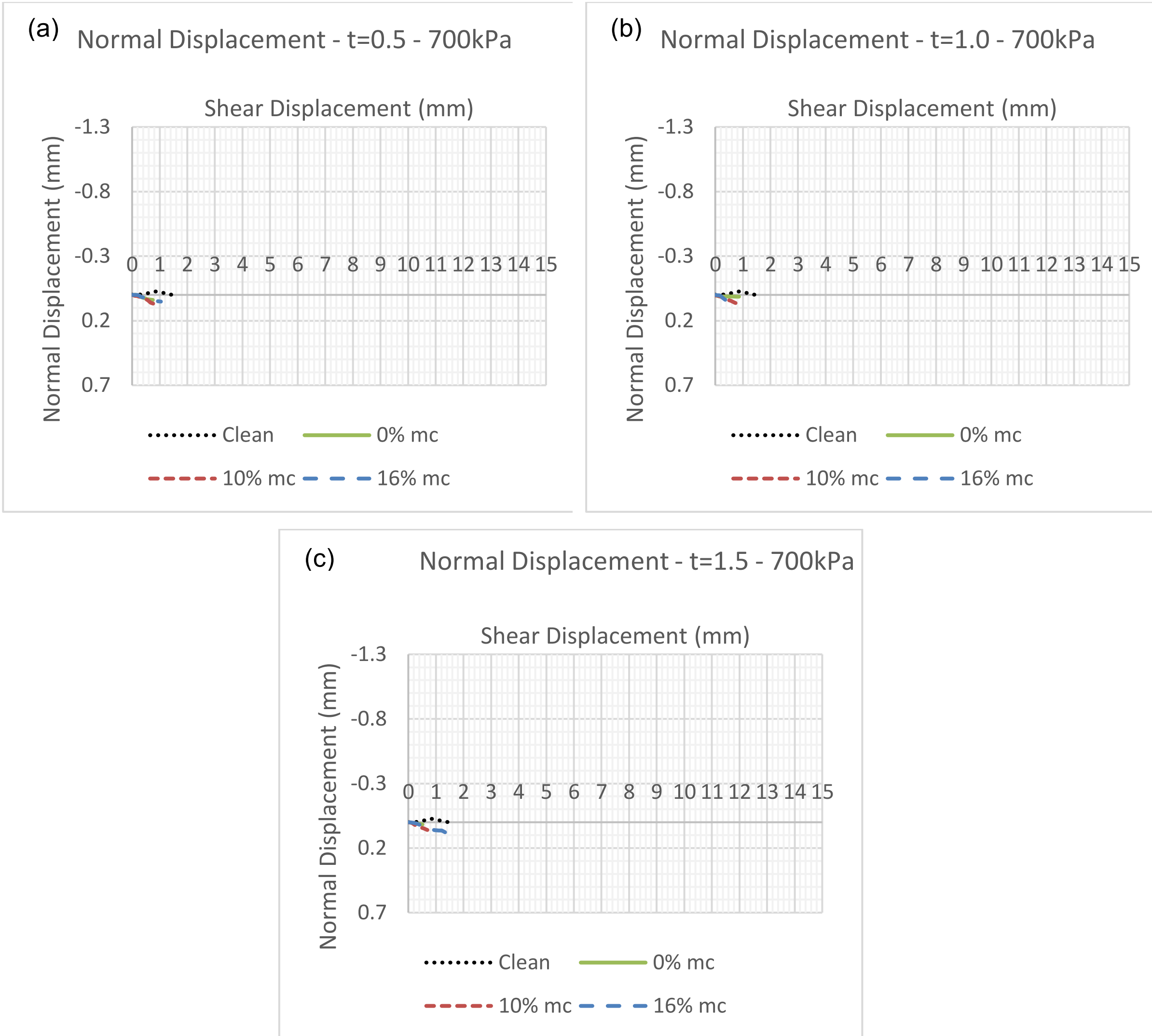

Figure 25: Shear Testing Results for 700 kPa (Shear Displacement vs Normal Displacement)a) t/a=0.5 b) t/a=1.0 c) t/a=1.5

CONCLUSIONS

This project analysed the effects of sodium bentonite clay infill at 0%, 10% and 16% moisture conditions, and various infill layer thickness, on the shear strength characteristics of clean and infilled rock joints subjected to Constant Normal Loading (CNL) boundary conditions. The key findings concluded that, while the various samples did show different behaviour based on moisture content, the magnitude of the shear strength of the rock joints was not dictated by the moisture content but rather: the joint profile when the infill thickness was less than the asperity height (highest in clean rock joints), the joint profile and infill material shear strength properties when the infill thickness was equal to the asperity height, and infill material shear strength properties principally controlled when the infill thickness was greater than the asperity height. The rate of convergence of the shear stress, normal stress and normal displacement trends increased as the moisture content increased when subjected to lower applied normal pressure of 100 and 300 kPa. With decreased moisture content, the angle of internal friction of the infill increased, resulting in increased friction forces acting on the infill soil particles. However, when the moisture content increased the soil particles could more easily slide past each other facilitating decreased variance in shear stress, normal stress, and normal displacement (including increased contraction). This decreased variance allows the material to achieve equilibrium more readily with the trending magnitude of the shear stress, normal stress or normal displacement. Due to the limiting loading of 2200 N of the direct shear apparatus used, the data at higher applied normal loadings is generally inconclusive.

Actual outcomes include: higher shear strength of clean rock joints in comparison to clay infilled joints, when the infill thickness was less than the asperity height the asperity profile of the artificial rock joint controlled the behaviour of the sample, at higher thicknesses (infill thickness equal to or greater than asperity height) the infill material predominantly controlled the behaviour of the joint, and that as the moisture content increased the magnitude of the converging trend remained largely unaffected. The key finding of this project is that as the moisture content increased, the rate of convergence of the shear stress, normal stress and normal displacement trends increased at lower applied normal loading conditions. However, due to the limiting loading of 2200 N of the direct shear apparatus used the data at higher applied normal loadings is generally inconclusive. Due to limited data set at higher applied normal stresses, further research should investigate the impact of variable moisture conditions on the shear strength properties of clay infilled rock joints by narrowing the scope to include more moisture content conditions tested at relatively low applied normal stresses (up to 300 kPa). In addition, testing under CNS conditions should be investigated to determine the behaviour on deeper rock joints.

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